TRIANGLE MESH CONCRETE REINFORCEMENT

ENGINEERS' HANDBOOK

AMERICAN STEEL & WIRE CO.

C. N. Little

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American Steel & Wire Co.

Handbook and Catalogue

of

Concrete Reinforcement

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CONTENTS

INTRODUCTION

In presenting this Revised Edition of our Engineers' Handbook and Catalogue on Triangle Mesh Reinforcement for Concrete we have attempted to touch briefly not only the reinforcement for concrete but concrete itself. The data contained herein is the result of careful study by our Engineering Department, and also by some of the best known and accepted authorities.

Many paragraphs and chapters dealing with Reinforced Concrete are selected and reprinted by permission from text books, bulletins, and other publications, with due mention of the source and authority in each instance.

Tables giving the weights and fabric per square foot, number, sizes, spacing and areas of wires and longitudinal strands of Triangle Mesh Woven Wire Reinforcement are given on pages 110, 111 and 112, and tables of Resisting Moments, Thicknesses of Slabs, Diagrams, etc., occur on pages 74 to 89, inclusive.

By the use of Tables 1 to 12, inclusive (No. 1 being found on page 74), giving the resisting moments of reinforced concrete slabs, suitable fabric may be selected to use either with or without bars for all kinds of loads and spans and conditions met with in the construction of buildings, bridges and reinforced concrete in general. In these tables we have assumed different allowable stresses in the steel, namely, 16,000, 18,000, and 20,000 pounds per square inch, depending on its elastic limit, and different proportions of cement, sand and stone in the concrete. We, however, recommend in general using a 1:2:4 mixture for floors or other structures subjected to high stresses.

While we recommend the use of a fabric made with a mild steel we can furnish it in any grade of steel desired.

Triangle Mesh Woven Wire Reinforcement for Concrete is made with either solid or stranded longitudinal members, properly spaced by means of diagonal or cross wires so arranged as to form a series of triangles between the longitudinal or tension members; the longitudinal members being invariably spaced 4 inches apart, the cross wires either 2 inches or 4 inches, as desired, providing either a 2-inch or 4-inch mesh. The sizes of both longitudinals and cross wires are

varied in order to provide the cross sectional areas of steel required to meet the conditions.

Triangle Mesh Reinforcement, we believe, is the most efficient material on the market for the purposes:

It provides a more even distribution of the steel, reinforcing in every direction.

Tension of carrying members accurately spaced.

A most perfect mechanical bond.

When a specific size of fabric or area of steel is specified it is impossible to leave out any portion of the reinforcement.

Minimum cost of installation.

Easily handled and stored on the work.

Low cost of inspection.

An absolutely continuous action from one end of the structure to the other.

Higher elastic limits with the same quality of steel due to cold drawing.

Every ounce of steel is tested, as it cannot be cold drawn without showing defects, if any.

Distributes the stresses due to a concentrated load over a greater area.

Triangle Mesh Reinforcement is the only design of woven wire fabric in which the cross or diagonal wires assist the longitudinal or tension members in carrying the load.

While reinforcing fabrics are made both galvanized and not galvanized, we strongly recommend the latter, due to the fact that a much better adhesive bond is provided and also greater strengths. In the case of a galvanized wire the adhesion between the reinforcement and the concrete is to the coating on the steel and not to the steel itself, and also in the galvanizing process the steel is annealed or softened, thereby reducing its elastic limit and ultimate strength.

It is a well-known fact that steel thoroughly imbedded in a proper mixture of concrete does not rust, and in the case of a smooth round rod used as reinforcement it is more desirable to have a thin surface coat of rust, than if it were perfectly bright and smooth, provided the rust has not penetrated sufficiently far to pit the steel and produce a scale. This slight coating of rust provides a rougher surface and therefore a better bond.

AMERICAN STEEL & WIRE COMPANY.
CONCRETE REINFORCEMENT DEPT.

REINFORCED CONCRETE.

In dealing with the uses and properties of reinforced concrete, we reprint, by permission, Chapter I, from "Reinforced Concrete," by Buel & Hill:

CHAPTER I.—ECONOMIC USE AND PROPERTIES OF REINFORCED CONCRETE.

Concrete alone, considered as a building material, is nothing more nor less than a kind of masonry. The distinguishing features between rubble masonry and concrete are really confined to the methods of mixing and placing the materials. The results obtained with rubble masonry made of very small stone and with concrete made of large stone would be practically identical . The old Roman concrete was made with large stones, and may be classified either with rubble or concrete masonry. The value of either rubble or concrete as a material for construction depends largely on the quality of the cement used and the care exercised in the mixing and placing. Examples of masonry structures composed of large stones reinforced or tied together with iron rods and bars are found in the works of all periods, but usually only in connection with cut-stone masonry. The cost of such reinforcement was very great compared with the additional strength secured, and with rubble masonry the mechanical difficulties involved and the comparative cost render it impracticable.

Reinforced Concrete..—With the advent of modern concrete the facilities with which reinforcing rods or bars of metal may be embedded anywhere in the mass of the masonry was soon seen and taken advantage of. The compressive resistance of concrete is about ten times its tensile resistance, while steel has about the same strength in tension as in compression. Volume for volume steel costs about fifty times as much as concrete. For the same sectional areas steel will support in compression thirty times more load than concrete, and in tension three hundred times the load that concrete will carry. Therefore, for duty under compression only, concrete will carry a given load at six-tenths of the cost required to support it with steel. On the other hand, to support a given load by concrete in tension would cost about six times as much as to support it with steel. These economic ratios are the *raison d'etre* of reinforced concrete . If the various members of a structure are so designed that all of the compressive stresses are resisted by concrete and steel is introduced to resist the tensile stresses, each material will be serving the purpose for which it is the cheapest and best adapted and one of the principles of economic design will be fulfilled.

Other important advantages secured in the combination of concrete and embedded steel are that the protection of the metal elements from corrosion is practically perfect; that, with properly selected ingredients, the fire and heat resisting qualities are very high, perhaps surpassed by no other building material except fire-brick; and, in many

cases, that the substantial appearance of a masonry structure is obtained at about the cost of a more or less temporary unprotected steel structure. When intelligently reinforced with steel, concrete becomes a material suitable and economical for beams, floors, and long columns, tanks, reservoirs, conduits, and sewers; admirably adapted to arch construction, and often economical for dams and retaining-walls. Even in concrete that is not subjected to tension or flexure it is often desirable to introduce steel reinforcement to prevent the occurrence of cracks due to shock or settlement, or other causes.

Properties of Concrete.—A knowledge of the properties of materials is the first requisite for safe and economic designing of structures. The properties of reinforced concrete comprise not only those of the concrete and of the steel elements considered separately, but may be said to include those properties or characteristics of the composite mass that control the distribution of stresses between the elements of the combination of units and determine the nature of their inter-relation. Such properties as are required by the practical engineer or architect in intelligent designing are here assembled in concise form, with values assigned to them that are considered to be safe and conservative deductions from the most recent experiments accessible. The scope and purpose of this work does not permit of an elaborate exposition of all the recent experiments nor of an exhaustive discussion of the deductions to be drawn therefrom.

Portland-cement concretes only will be considered. Concrete made with natural slag, or Puzzolanic cements, although adapted to many uses, do not possess the qualities desirable for reinforced concrete structures, and all the experiments known to the writer, on which the theories of reinforced concrete are based, have been with Portland-cement concretes. The object of reinforcing concrete with steel is to secure greater strength or safety, or both, than can be attained with concrete alone; and excepting a few special cases where the concrete is used principally for a filling or to add mass to the construction, concrete made with Portland cement will generally be found the most economical for equal strength, safety and durability.

The properties of concretes vary with their age and with the proportions and quality of the ingredients. The values given here are for concretes made with (1) true Portland cement having a tensile strength per square inch neat, in 7 days of 450 to 650 lbs., and in 28 days of 540 to 750 lbs.; (2) silica sand, not necessarily sharp nor coarse, but absolutely clean, and preferably a mixture of fine and coarse; and (3) good, hard, screened broken stone or clean gravel. The proportions of cement to sand generally used in the mortar or matrix, and for which there are reliable experimental data, vary from 1 of cement to 1 of sand up to 1 of cement and 6 of sand; and the proportion of mortar or matrix to the aggregate (broken stone or gravel) is from 100 to 110 per cent of the voids of the latter.

This method of specifying the proportions, by cement to sand in the mortar or matrix and by mortar or matrix to voids in the aggregate.

is here adopted because it is believed that the ratio of matrix to aggregate, where the latter is good clean material, does not affect the strength of the concrete, except in so far as sufficient matrix should be provided to fill the voids in the aggregate. Other things being equal, the strength of the concrete will be proportional to the strength of the mortar, and the maximum strength for a given matrix or mortar will be attained when all voids are filled . In practice this requires a volume of matrix about 10 per cent. in excess of the voids in the aggregate. Thus, if by mixing several sizes of broken stone or gravel, the proportion of voids to be filled is reduced from 45 per cent. or 50 per cent. down to 30 per cent., the proportion of matrix, cement and sand, to aggregate may be considerably reduced without reducing the strength of the concrete or affecting its properties. Where cement or sand are dear and stone and gravel are cheap advantage may be taken of this method to reduce the cost of the concrete very materially.

The values here given are for concretes seven days, and one, three, and six months old. Those values should be used which correspond to the age at which the structure may be subject to its full load.

Compressive Strength.—Concrete is more often used in compression than in any other way, since it is more economical and has heretofore been considered more reliable under compressive strains than under transverse or tensile strains. Until very recent years engineers and architects hardly gave serious consideration to the value of concrete as a material to resist bending or tensile stresses, but at the present time comparatively few hesitate to use it in beams and similar situations where it is partly subjected to tensile stress, and considerable number of eminent members of both professions have constructed works where the tensile strength of the concrete is taken advantage of. The best practice, where any tensile strains can occur, is to reinforce the section with steel. The two chief factors that determine the compressive strength of a concrete are its age and the proportion of sand to cement in the matrix. The quality of the cement, sand, and aggregate have more or less influence on the resulting concrete, but with any good brand of modern high-burned Portland cement, clean sand, and clean, hard stone, substantially the same results may be secured. Factors of far greater weight are the manner and conditions of mixing and placing, and the personal equation of the operator. On this account it is extremely difficult to harmonize or draw conclusions from the large number of isolated tests that have been made by independent investigators under widely varying conditions and often with different objects in view.

A set of experiments made at the Watertown Arsenal for Mr. George A. Kimball, Chief Engineer of the Boston Elevated R. R., in 1899, are the most homogeneous and systematic set of tests that have as yet been published, and are given in Table I.

From these tests Mr. Edwin Thatcher has deduced formulas for the ultimate strength of concretes. They give results that agree with

the average of the experiments and can be entirely relied upon for concretes carefully made from good materials. They are as follows: The ultimate compressive strength in pounds per square inch of concrete:

$$\begin{array}{ll} \text{7 days old} & =1{,}800-200\left(\dfrac{\text{volume of sand}}{\text{volume of cement}}\right), \\ \text{1 month old} & =3{,}100-350\left(\qquad\text{do.}\qquad\right), \\ \text{3 months old} & =3{,}820-460\left(\qquad\text{do.}\qquad\right), \\ \text{6 months old} & =4{,}900-600\left(\qquad\text{do.}\qquad\right), \end{array}$$

These formulas give the results shown in Table II.

Tensile Strength.—The tensile strength may be safely placed at one-tenth of the compressive strength, and the modulous of transverse rupture, $f=\frac{M}{S}$ at about $1\frac{6}{10}$ that of the tensile strength. Tetmajer gives the ratio as follows for Portland-cement mortars consisting of 1 of cement to 3 of sand by weight:

$$\text{Tensile strength}=\left(\frac{\text{compressive strength}}{8.64+1.8\text{ log. of age in months}}\right).$$

Shearing Strength.—M. Mesnagen states that the shearing strength of concrete is from 1.2 to 1.3 times the tensile strength. Bauschinger gives the shearing strength of concrete four weeks old at 1.25 times the tensile strength, and at two years old 1.5 times the tensile strength. A paper on the "Shearing Resistance of Reinforced Concrete," by S. Zipkes, translated by Mr. Leon S. Moisseiff, in "Cement," for March, 1906, gives the average shearing strength, at the appearance of the first cracks, at 81 lbs. per square inch. At the time of rupture, he found the average to be 357 lbs. per square inch. Prof. Moersch ("Cement", July, 1893) obtained an average shearing resistance of 400 to 440 lbs. per square inch. Prof. Moersch's beams were tested at three months old, whereas Mr. Zipkes' specimens were all tested at an age of 50 days. Considering the difference in the age of the specimens, the agreement is fair.

TABLE I. — SHOWING COMPRESSIVE STRENGTH OF CONCRETE AS DETERMINED BY TESTS MADE AT WATERTOWN ARSENAL IN 1899.

MIXTURE 1 : 2 : 4.

Brand of Cement.	Compressive Strength, Pounds per Square Inch.			
	7 Days.	1 Month.	3 Months.	6 Months.
Atlas	1,387	2,428	2,966	3,953
Alpha	904	2,420	3,123	4,411
Germania	2,219	2,642	3,082	3,643
Alsen	1,592	2,269	2,608	3,612
Average	1,525	2,440	2,944	3,904

MIXTURE 1 : 3 : 6.

Brand of Cement.	Compressive Strength, Pounds per Square Inch.			
	7 Days.	1 Month.	3 Months.	6 Months.
Atlas	1,050	1,816	2,538	3,170
Alpha	892	2,120	2,355	2,750
Germania	1,550	2,174	2,486	2,930
Alsen	1,438	2,114	2,349	3,026
Average	1,232	2,063	2,432	2,969

MIXTURE 1 : 6 : 12.

Brand of Cement.	Compressive Strength, Pounds per Square Inch.			
	7 Days.	1 Month.	3 Months.	6 Months.
Atlas	594	1,090	1,201	1,583
Alpha	564	1,218	1,257	1,532
Germania	759	987	963	815
Alsen	417	873	844	1,323
Average	583	1,042	1,066	1,313

TABLE II.—SHOWING ULTIMATE COMPRESSIVE STRENGTH OF CONCRETE AS DETERMINED BY THACHER'S FORMULAS.

Mixture.	Age.			
	7 Days.	1 Month.	3 Months.	6 Months.
1:1 :3	1,600	2,750	3,360	4,300
1:2 :4	1,400	2,400	2,900	3,700
1:2½:5	1,300	2,225	2,670	3,400
1:3 :6	1,200	2,050	2,440	3,100
1:3½:7	1,100	1,875	2,210	2,800
1:4 :8	1,000	1,700	1,980	2,500
1:5 :10	800	1,350	1,520	1,900
1:6 :12	600	1,000	1,060	1,300

TABLE III.—SHOWING MODULUS OF ELASTICITY OF CONCRETE AS DETERMINED BY TESTS AT WATERTOWN ARSENAL IN 1899.

MIXTURE 1 : 2 : 4.

Brand of Cement.	Modulus of Elasticity between Loads of 100 to 600.			
	7 Days.	1 Month.	3 Months.	6 Months.
Atlas	2,778,000	3,125,000	4,167,000	3,125,000
Alpha	....	2,083,000	4,167,000	3,125,000
Germania	2,500,000	....	3,571,000	4,167,000
Alsen	2,500,000	2,778,000	2,778,000	4,167,000
Average	2,592,000	2,662,000	3,670,000	3,646,000

MIXTURE 1 : 3 : 6.

Brand of Cement.	Modulus of Elasticity between Loads of 100 to 600.			
	7 Days.	1 Month.	3 Months.	6 Months.
Atlas	1,677,000	3,125,000	2,778,000	3,571,000
Alpha	....	2,083,000	3,571,000	4,167,000
Germania .. .	2,273,000	2,273,000	2,778,000	3,125,000
Alsen	1,667,000	2,273,000	2,778,000	3,571,000
Average ..	1,869,000	2,438,000	2,976,000	3,608,000

MIXTURE 1 : 6 : 12.

Brand of Cement.	Modulus of Elasticity between Loads of 100 to 600.			
	7 Days.	1 Month.	3 Months.	6 Months.
Atlas	....	1,316,000	1,136,000	1,786,000
Alpha	....	1,667,000	1,786,000	1,923,000
Germania .. .	....	961,000	2,083,000	1,786,000
Alsen	....	1,562,000	1,562,000	1,786,000
Average ..	....	1,376,000	1,642,000	1,820,000

Modulus of Elasticity.—It has been said that no property of materials of construction is as uniform and reliable as the modulus of elasticity. This may be true of the modulus of elasticity of concrete, but the great variation in its value, as determined by the experiments heretofore published, has left the matter very much in the dark. Its value has been stated all the way from 750,000 to 5,000,000. This has been a discouraging condition for conservative constructors, and, no doubt, has greatly retarded the introduction of reinforced concrete in important works. The Watertown Arsenal tests in 1899 give values for the modulus of elasticity E of concrete as shown in Table III.

From Table III the following formulas have been deduced, giving values very close to the averages of the experiments and sufficiently exact for all practical purposes. For concrete:

$$\text{7 days old,} \quad E=2{,}600{,}000-700{,}000\left(\frac{\text{volume of sand}}{\text{volume of cement}}-2\right),$$
$$\text{1 month old,} \quad E=2{,}900{,}000-300{,}000\left(\text{do.}\quad -1\right),$$
$$\text{3 months old,} \quad E=3{,}600{,}000-500{,}000\left(\text{do.}\quad -2\right),$$
$$\text{6 months old,} \quad E=3{,}600{,}000-600{,}000\left(\text{do.}\quad -3\right),$$

If the term $\left(\frac{\text{volume of sand}}{\text{volume of cement}}-c\right)$ is zero or less than zero (negative), the entire term is to be considered zero. In other words, all negative values must be considered as zero. Table IV shows the moduli of elasticity as determined by the above formulas. These values are sufficiently reliable for all ordinary purposes, and are probably as

near to the truth as any that can be deduced from the experiments at present available. A large number of carefully executed experiments will be required to determine these values with greater precision.

TABLE IV.—SHOWING MODULI OF ELASTICITY OF CONCRETE AS DETERMINED BY FORMULAS.

Mixture.	Age.			
	7 Days.	1 Month.	3 Months.	6 Months.
1:1 :3	2,600,000	2,900,000	3,600,000	3,600,000
1:2 :4	2,600,000	2,600,000	3,600,000	3,600,000
1:2½:5	2,250,000	2,450,000	3,350,000	3,600,000
1:3 :6	1,900,000	2,300,000	3,100,000	3,360,000
1:3½:7	1,550,000	2,150,000	2,850,000	3,300,000
1:4 :8	1,200,000	2,000,000	2,600,000	3,000,000
1:5 :10	500,000	1,700,000	2,100,000	2,400,000
1:6 :12	....	1,400,000	1,600,000	1,800,000

Mr. W. H. Henby has given forty-eight determinations of the modulus of elasticity under tensile stress and eighteen under compressive stress, but the conditions were varied so that they can only be compared in groups of two or three tests with constant conditions, and as would naturally be expected, the results were very erratic and are not conclusive. Prof. Wm. H. Burr concludes that the same values may safely be used for the modulus of elasticity in tension as in compression.

The values of E are only given for loads between 100 and 600, since these limits include the practical range of safe working stresses per square inch. For purposes of computing the ultimate strength, which would be for loads from 600 to 4,000 lbs., E would have considerably lower values. For loads between 1,000 and 2,000 lbs. the values would be from one-half to two-thirds of those given for loads between 100 and 600 lbs. For loads over 2,000 lbs. satisfactory data are not known to the writer. Table V gives values of the modulus of elasticity for stresses up to 2,000 lbs. per square inch as determined at the Watertown Arsenal in the series of tests made for Mr. Geo. A. Kimball, Chief Engineer of the Boston Elevated Railroad, in 1899. These determinations show that the modulus of elasticity is very much less at stresses between 1,000 and 2,000 lbs. per square inch than between 100 and 600 and 1,000 lbs. per square inch, but they are not sufficiently comprehensive to form the basis of any satisfactory rule or formula for the ratio of the modulus of elasticity to the stress per square inch.

TABLE V.—SHOWING REDUCTION IN VALUE OF E_c WITH INCREASING LOADS. VALUES GIVEN ARE THE MEAN OF THOSE FOR SEVERAL EXPERIMENTS WITH SEVERAL STANDARD BRANDS OF PORTLAND CEMENT.

Age.	Concrete 1 . 2 . 4.			Concrete 1 . 3 . 6.		
	100-600	100-1,000	1,000-2,000	100-600	100-1,000	1,000-2 000
7 days.....	2,592,000	2,053,000	1,351,000	1,869,000	1,529,000	
1 mo......	2,662,000	2,444,000	1,462,000	2,438,000	2,135,000	1,219,000
3 mos.....	3,670,000	3,170,000	2,157,000	2,976,000	2,656,000	1,805,000
6 mos.....	3,646,000	3,567,000	2.581.000	3,608,000	3,503,000	1,868,000

Professors Boeck and Melan found a value of E at about 750,000 in connection with the Austrian experiments, where a number of arches were tested to destruction. In calculations of ultimate strength by formulas, assumed values of E ranging from 1,500,000 to 750,000, according to the mixture, age, and the ultimate load per square inch. would seem to agree more nearly with the average of previous experiments than values of E corresponding to loads much less than the ultimate strength.

Two important points to be noted in connection with this subject are that the elastic limit of concrete, so far as it has been determined, is very close to the ultimate strength, and that its stress-strain diagram is a curve, instead of being practically a straight line as it is with steel inside of the elastic limit . The nature of this curve cannot be determined from the limited number of determinations that have been published.

Working Loads.—In Table VI are given what are considered safe working loads, in pounds per square inch, and properties for concretes in which the mortar or matrix is 1 of cement to 2 of sand and 1 of cement to 3 of sand, and in which all the voids in the aggregate are filled. According to present practice, these mixtures will about cover the range for reinforced concrete.

Properties of Steel.—The following properties of steel for use in computing reinforced concrete sections, with the values assigned to them, will be used herein. These values are believed to be safe, but may be varied as conditions require, according to the judgment of the designer:

Ultimate strength, 58,000 to 66,000 lbs. per square inch.

Elastic limit, 55 per cent. of the ultimate strength.

Modulus of elasticity, 29,000,000.

Working stress, factor of 4, 15,000 lbs. per square inch.

Working stress, factor of 5, 12,000 lbs. per square inch.

Rate of expansion per degree Fahrenheit, 0.00000648 to 0.00000686.

Relations Between Concrete and Steel.—The character of the relations that exist between the concrete and steel elements of reinforced concrete combinations depends first on the design of the section. If the

two elements act independently in resisting the stresses, so that either the one or the other might carry all the load, it may be called a *composite design.*

If some of the forces are resisted entirely by the steel and other forces resisted entirely by the concrete, so that if the element resisting one force failed the entire section would fail, it may be called a *combination design.*

If the disposition of the steel and the concrete in the section is such that the two elements act as a single unit, all stresses being divided between the concrete and the steel, where the latter occurs, and that the entire omission of the steel would only result in reducing the strength of the section, it may be called a true *monolithic design.*

While many composite designs have been loosely classed with "concrete-steel," they really have little in common with the combination and monolithic designs. Since the concrete and the steel are independent of each other, and either one may carry all the load, it is clear that each element should be calculated independently and like an all-concrete or an all-steel section, as the case may be. This is not to

TABLE VI.—SHOWING SAFE WORKING STRESSES FOR CONCRETE.

Mixture.	1 to 2 Matrix.				1 to 3 Matrix.			
Age.	1 Month		6 Months.		1 Month.		6 Months.	
Safety factor	6	5	6	5	5	5	5	5
Compression, lbs.*.	400	500	600	700	340	400	500	600
Tension, lbs.......	40	50	60	70	35	40	50	60
$f=\frac{M}{S}$	64	80	96	112	56	64	80	97
Shearing	50	62	75	87	44	50	62	75
E	2,600,000		3,600,000		2,300,000		3,360,000	

Rate of expansion per degree Fahrenheit	(Clark)	.00000795
	(Rae and Dougherty)....	.00000655 for 1 : 3 : 5 concrete
	(Rae and Dougherty)....	.00000561 for 1 : 2 mortar
Adhesion to iron or steel metalic surface, ultimate	(Bauschinger)	570 to 640 pounds per square inch
	(Hatt).................	636 to 756 pounds per square inch
Safe working adhesion......................		60 to 100 pounds per square inch

* These values for compression are intended for use with the straight-line formulas only. For the formulas of the parbolic type they should be reduced, as the latter give larger moments of resistance (M_o) than the straight-line formulas for the same value of compression in the extreme fibers (f_c').

Note.—Prof. Hatt also found that the friction of smooth round rods embedded in concrete after they started to slip was from 50 per cent. to 70 per cent. of the adhesion.

For concrete not reinforced with steel, use two-thirds the values given in the tables for tension and $f=M \div S$.

imply that the concrete may not stiffen the steel and prevent it from buckling, but as they do not act together as a combination or unit, and as the steel does not reinforce the concrete, except in the manner that any additional and independent section may reinforce another, designs of this type should scarcely be classed with concrete steel or reinforced concrete.

Combination designs include concrete-steel beams after the concrete on the tension side has been strained beyond the point of rupture, which will occur in a well-designed beam long before the ultimate strength of the beam is reached. Concrete beams reinforced with steel, under loads that produce maximum tensile stresses in the concrete less than the ultimate strength, act as a single unit and may be classed as monolithic.

The most important characteristics or properties required to determine the distribution of stresses between the concrete and steel are the relations existing between the following:

A_c=area of the section of the concrete.

A_s=area of the section of the steel.

E_c=the modulus of elasticity of the concrete.

E_s=the modulus of elasticity of the steel.

Under direct compression or tension the stresses will be distributed between the two elements in the proportion of $F_c : F_s :: A_c E_c : A_s E_s$, where

F_c=the total stress in the concrete

and

F_s=the total stress in the steel.

From this is derived the equation

$$F_s = F_c \frac{A_s E_s}{A_c E_c}, \qquad (1)$$

and if $f_c = \frac{F_c}{A_c}$ = the stress per square inch in the concrete and $f_s = \frac{F_s}{A_s}$ = the stress per square inch in the steel, we have

$$f_s = f_c \frac{E_s}{E_c}, \qquad (2)$$

which is to say that the stress per square inch in the two elements is directly proportional to their respective moduli of elasticity. This is derived directly from the definition of the modulus of elasticity which is the ratio of the stress per unit of section to the deformation. When the modulus of elasticity for steel is stated to be 29,000,000, it means that one pound per square inch tension or compression will stretch or compress the section an amount equal to its length divided by 29,000,000, and if E_c, for the concrete, is 1,933,333, one pound per square inch will stretch or compress it an amount equal to its length divided by 1,933,333. If $\frac{E_s}{E_c} = \frac{29{,}000{,}000}{1{,}933{,}333} = 15$, and if the same intensity of stress per square inch exists in both the concrete and the

steel, the concrete will be deformed fifteen times as much per unit of length as the steel, or in the ratio $\frac{E_s}{E_c}$. If, however, the stress per square inch in the steel is fifteen times that in the concrete, or in the ratio of $E_s : E_c$, then the deformation will be the same per unit of length in both. Unless this latter condition maintains in every part of a concrete and steel structure of any description the surfaces of the two elements in contact will slide over each other or the concrete near the steel element will be strained beyond its elastic limit or its ultimate resistance.

While it is the invariable practice to meet this condition in the design of arches, columns, etc., concrete-steel beams are quite generally designed on the theory that the steel does all the work on the tensile side of the neutral axis. There is no doubt whatever that the concrete on the tensile side of a well-designed reinforced concrete beam will fail long before the ultimate strength of the beam is reached, since most all of the tests to destruction have demonstrated it to be so. This theory will be treated at some length in the chapter on beams.

Coefficient of Expansion.—The thermal changes in reinforced concrete have ceased to be a matter for discussion from a practical viewpoint, and have been relegated to the laboratories for the determination of the last decimal in the rates of expansion. Some of the most recent and reliable determinations made by Rae and Dougherty at Columbia University and by Prof. W. D. Pence at Purdue University gave the rate of expansion for Portland-cement concrete with various proportions of sand and stone or gravel, such as are generally used in practice, at 0.00000545 to 0.00000655 per degree Fahrenheit. The later value by Rae and Dougherty is perhaps the more reliable, as the experiments were conducted with great care. Clark gives the rate at 0.00000795, which averaged with the mean of Prof. Pence's determination, 0.00000545, gives 0.00000670. This is less than 2½ per cent. greater than the value given by Rae and Dougherty.

The rate of expansion per degree Fahrenheit for wrought-iron and steel is given by Kent at 0.00000648 to 0.00000686, and by U. S. Reports on Iron and Steel at 0.00000617 to 0.00000676. The mean of these is about 0.00000657. From this it appears that the difference in the rate for concrete and for steel is only a fraction of 1 per cent.

Aside from this the large number of reinforced concrete structures that have been exposed to the weather in severe climate for years without any indication of injurious effect from thermal changes is a sufficient proof that if there is any difference in the rate for the two materials, it is not enough to be of consequence.

Adhesion Between Concrete and Steel.—Next in importance to the ratio between the stress per square inch and the moduli of elasticity is the adhension between the concrete and the steel. Table VI gives the ultimate and safe working values of this property in pounds per square inch. In the design of any combination or monolithic member of reinforced concrete the bond between the two elements is of vital impor-

tance. In the majority of cases met in practice, the relation between the elements is such that the entire stress in the steel must be transmitted to it by this bond of adhesion. When the shear per foot run between the steel and concrete exceeds the safe working adhesion, resort must be had to a mechanical bond. Various devices have been used to obtain an effective bond, such as corrugating or twisting square or flat rods or bars, driving rivits in flat bars, the projecting heads of which serve the purpose, and deforming round rods so that they are made up of alternate round and flat sections but with the same sectional area at every point.

Some engineers have objected to the use of square or flat sections on the ground that the sharp re-entering angles formed in the concrete weaken the latter and induce cracks to start from the angle when subjected to loads or shocks. In cast-iron, a material that has several properties similar to those of concrete, re-entering angles greatly weaken the sections, and therefore castings are generally boldly filleted at such angles. The writer does not know of any tests that throw light on this question, but notwithstanding the fact that considerable concrete has been reinforced with square and flat steel, it would seem to be safer and conservative practice to avoid all sharp re-entering angles in concrete. By far the larger part of all the reinforced concrete in Europe has been made with round rods or wires In some cases steel angles, I-beams, or T's have been used, but squares and flats, if used at all, do not seem to have met with general favor Tests more recently made in America indicate a considerable gain in ultimate strength of reinforced-concrete beams when rods are used that give a mechanical bond, as compared with beams made with plain rods.

As this second edition is just going to press, reports on the effect of the California earthquake on buildings of different types of construction are just beginning to come in. These are as yet too meagre to form the basis of any conclusion. It is worthy of note, however, that the buildings with steel frames have stood the test very well, and that, of the Leland Stanford University Buildings, at Palo Alto, the damage was confined almost entirely to those with brick or stone masonry walls, while some buildings with monolithic concrete walls, not reinforced, escaped with little or no injury. Some of these buildings had concrete floors, reinforced with twisted rods, which are reported to have stood the test satisfactorily. It would seem to be prudent, in designing reinforced concrete buildings, in localities subject to earthquake, to plan the reinforcing steel members so that they would be everywhere tied together and of such strength that they would be in stability without assistance from the concrete.

The late Mr. Geo. S. Morrison, in an address approving the principle of reinforced concrete, referred to such construction as "concrete structures with metal structures inside." If the writer interprets this correctly, Mr. Morrison referred to structures in which the metal elements alone would form a complete and stable structure.

though not necessarily one of sufficient strength to carry the required loads. This conception of a reinforced concrete structure seems to the writer to be the correct one, but, of course, it is not the cheapest that can be built. Many errors are made in attempting to keep down first cost, and such errors enter into a larger proportion of the structures built during the early stages of the development of any new method or system of construction than they do afterwards. As an example of this, all of our early metal bridges in the United States were built too light, even for the loads then in vogue, and we have come to adopt much heavier details than would previously have been used for the same duty. The writer believes that this applies with a special force to reinforced-concrete construction and that the development of design will tend toward the idea of making the embedded metal parts at least capable of supporting themselves in their position in the structure without assistance or connections from the concrete in which they are to be embedded. Of course, this does not refer to all kinds of structures, but more especially to reinforced-concrete buildings. The Melan system of arch construction is a good illustration of this idea, as its reinforcement consists of a perfect metal arch, which is, in stability, without any assistance from the concrete and is sometimes made sufficiently strong to carry the entire load.

COSTS.

In considering a material to be used in building, one of the first things that an owner asks is, "What is the cost," and usually this means the first cost, forgetting to consider the most important factor, the cost of maintenance and repairs, and the insurance, which runs on year after year.

The first is practically the whole cost in using Reinforced Concrete, as compared to other building materials, and this varies according to the character of the construction and the purchase price of materials.

The following paragraphs selected from Page 24 of Taylor & Thompson's volume, "Concrete Plain and Reinforced," contain some interesting data:

APPROXIMATE COST OF CONCRETE.

The cost of concrete depends more upon the character of the construction and the conditions which govern it than upon the first cost of the materials. In a very general way, we may say that when laid in large masses or in a very heavy wall, so that the construction of the forms is relatively a small item, the cost per cubic yard in place is likely to range from $4 to $7. The lower figure represents contract work under favorable conditions with low prices for materials, and the higher figure small jobs and inexperienced men. Similarly, we may say that for sewers and arches, where centering is required, the price may range from $7 to $14 per cubic yard. Thin building walls, under eight inches thick may cost from $10 to $20 per cubic yard, according to the character of construction and the finish which is given to the surface.

These ranges in price seem enormous for a material which is ordinarily supposed to be handled by unskilled labor, but it must be borne in mind that skilled workmen are required for constructing forms and centers, and often the labor upon these may be several times that of mixing and placing the concrete. As a rule, unless the job is a very small one or under the personal supervision of a competent engineer, it is cheaper and more satisfactory to employ an experienced contractor than day labor. Green men under an inexperienced foreman may not be counted upon to mix and lay over one-half the amount of concrete that will be handled by a skilled gang under expert superintendence.

A close estimate of cost may be reached, in cases where the conditions are known in advance, by taking up in detail and then combining the various units of the material and labor as outlined below.

Cost of Cement. As the price of Portland cement varies largely with the demand, it is necessary to obtain quotations from dealers for every purchase. It is such heavy stuff that the freight usually enters largely into the cost, and quotations should therefore be made f.o.b. the nearest point of delivery to the work. The cost of hauling by

wagon may be readily estimated by assuming that a barrel of cement weighs 400 pounds (gross), and that a pair of horses will haul over an average country road a load of, say 5,000 pounds, traveling in all a distance of 20 to 25 miles in a day, that is, 10 to 12½ miles with load. This assumes, of course, that the teams are good and properly handled.

Having found the cost of the cement per barrel, delivered, the approximate cost per cubic yard is at once obtained from the table on page 17. If, for example, the cost is $2 per barrel and proportions 1:2½:5 are selected, the cost of the cement per cubic yard of concrete will be 1.29×$2.00=$2.58.

Cost of Sand. The cost of sand depends chiefly upon the distance hauled. With labor at 15 cents per hour, the cost of loading (including the cost of the cart waiting at pit) may be estimated, if handled in large quantities, at 18 cents per cubic yard, or on a small job at 27 cents per cubic yard. For hauling add one cent for each 100 feet of distance from the pit. The additional cost of screening, if required, will vary with the coarseness of the material, but 15 cents per cubic yard may be called an average price for this, unless the sand is obtained by screening the gravel, when no allowance need be made. After finding the cost of one cubic yard of sand, the cost of the sand per cubic yard of concrete is readily figured from the table referred to. If, for example, the cost of sand screened, loaded and hauled 1,000 feet is 52 cents per cubic yard, the cost per cubic yard of concrete for proportions 1:2½:5 will be 0.45×$0.52=$0.23½.

Cost of Gravel or Broken Stone. If broken stone is used upon a small job for the coarse aggregate, it is usually purchased by the ton or cubic yard. A 2000-lb. ton of broken stone may be considered as averaging approximately 0.9 cubic yards, although differences in specific gravity cause considerable variation. A two-horse load is generally considered 1½ to 2 yards, the latter quantity requiring very high sideboards. The cost of screening gravel, if this is necessary, while a very variable item, may be estimated at 35 cents per cubic yard. The cost of loading gravel into double carts, with labor at 15 cents per hour, may be estimated on a small job at 38 cents per cubic yard. If handled in large quantities 25 cents is an average cost. The cost of loading, includes loosening and also the cost of the cart waiting at the pit. Hauling costs about one cent per cubic yard additional for each 100 feet of distance hauled under load. If, to illustrate, the cost of gravel picked, screened, loaded and hauled 1000 feet is 83 cents per cubic yard, the cost of the gravel per cubic yard of concrete for proportions 1:2½:5 will be 0.91×$0.83=$0.75½.

For distances up to 300 feet both sand and gravel can be hauled more economically by wheelbarrows than by teams. The cost of loading wheelbarrows is about half the cost of loading carts, while the cost of hauling with barrows per 100 feet is about four times greater.

Cost of Labor. With an experienced gang working at the rate of 15 cents per hour, the cost of mixing and laying concrete, if shoveled directly to place from the mixing platform, will average about 80 cents

per cubic yard, in addition to the work on forms. If, as is usually the case, the concrete is wheeled in barrows, 9 cents per cubic yard must be added to the above price for the first 25 feet that the barrows are wheeled under load, and 1¼ cents for each additional 25 feet wheeled. With other rates of wages, the cost may be considered as proportional. With a green gang, the cost will be nearly double the above figures, but as the men become worked in and organization perfected, the cost should approximate more nearly the prices given.

The labor on forms is not included in the above. This is an extremely variable item. The cost of building rough plank forms (not including cost of lumber) on both sides of a 5-foot wall may be as low as 14 cents per cubic yard of concrete, with other thicknesses of wall in inverse proportion. On elaborate work the price, which is really dependent upon the face area, may reach several dollars per cubic yard of concrete.

THE STRENGTH OF CONCRETE.

The strength of concrete varies (1) with the quality of the materials; (2) with the quantity of cement contained in a cubic yard of the concrete; and (3) with the density of the mixture.

We may say that the strongest and most economical mixture, consists of an aggregate comprising a large variety of sizes of particles, so graded that they fit into each other with the smallest possible volume of spaces or voids, and enough cement to slightly more than fill all of these spaces or voids between the solids of the aggregate. It is obvious that with the same aggregate the strongest cement will make the strongest concrete.

On important construction the various materials to be used should be carefully tested, and specimens of the mixture selected made up in advance and subjected to test. As a guide to the loads which concrete will stand in compression,—that is, under vertical loading where the height of the column or mass is not over, say, 12 times the least horizontal dimension,—we may give the following approximate figures as safe strengths, after the concrete has set at least one month, for the proportions which have previously been selected in this article as typical mixtures.

The figures, compared with the results of recent experiments on 12-inch cubes, allow a factor of safety of six at the age of one month, or eight at the age of six months, and are based on conservative practice. The relative strengths of the different mixtures are calculated from original investigations of the authors discussed in Chapter XIII.

Safe Strength of Portland Cement Concrete in Direct Compression.

Proportions	Pounds per square inch.	Tons per square foot.
1:2:4	410	29
1:2½:5	360	25
1:3:6	325	23
1:4:8	260	18

With a large mass foundation, take values one-eighth greater.
With a vibrating or pounding load, take one-half these values.

The tensile strength of concrete is very much less than the compressive strength. Experiments made by the authors, with mixtures of average proportions, give the ultimate fiber stress in beams as about one-eighth the breaking strength in compression.

STEEL FOR REINFORCING.

While there may or may not be advantages in using a high carbon, high tensile strength steel in reinforcing-concrete, the opinion in general seems to be in favor of a medium or mild steel. A tensile strength of 64,000 lbs. per square inch is about the minimum breaking point of ordinary mild commercial steel, while high carbon, high tensile strength steel will often run as high as 150,000 lbs. per square inch, and if used, less steel is required. But owing to the brittle nature of high carbon steel, as well as the difficulty in securing a uniform quality, it appears more dangerous to use.

The Coefficient or Modulus of Elasticity being one of the governing factors in reinforcing concrete, and this remaining the same in either a high or low carbon steel, it is usually more desirable to use a mild or commercial steel for reinforcing purposes.

We reprint below by permission on this subject, "Quality of Reinforcing Steel," from Page 291 "Concrete Plain & Reinforced," by Taylor & Thompson:

Quality of Reinforcing Steel.—It is generally recognized that in beam design the yield point of the steel shall be considered as the point of failure of this material in a reinforced beam. Tests show that when the metal reaches its yield point, the beam sags, and this deflection, due to the stretch of the steel, and in some cases to the slipping of the steel because of its reduced cross-section, is likely to produce crushing in the concrete.

The yield point of ordinary mild steel purchased in the open market, as determined by the drop of the beam in testing (the true elastic limit is several thousand pounds lower), cannot safely be fixed at a higher value than 30,000 pounds per square inch, although frequently, and in fact in the majority of cases, a value of at least 36,000 pounds and in many cases 40,000 pounds, will be found.

High steel, that is, steel containing a high percentage of carbon, has a much higher yield point than mild steel. If of first-class quality,* a minimum yield point may be placed at 50,000 or 55,000 pounds per square inch and much of it will reach 60,000 pounds. The ultimate strength should be not less than 105,000 pounds per square inch. Thus, if it can be safely employed in reinforced concrete, it is adapted to carry much higher stress than mild steel, and, conversely, a smaller percentage of it is required for the same moment of resistance. Many engineers do not approve of the use of high steel because of its brittleness, when of poor quality, and the danger of sudden accident, and because of the fact that it is prohibited in ordinary structural steel work.

Mild steel, that is, ordinary market steel, is manufactured and sold under such standard conditions that it may be safely used without test. High steel, on the other hand, must be very thoroughly tested. When tested, however, as per our specifications, page 38, it is entirely

*See Specifications for First-class Steel, p. 38.

safe and to be preferred to mild steel. The objection to it for reinforced concrete is based largely upon the use of a poor quality of material. Another objection which has been raised is that before the elastic limit is reached, the stretch in the high steel may produce an excessive cracking in the concrete in the lower portion of the beam, and thus expose the steel to corrosion. The mere fact that cracks are visible does not prove that they are dangerous, because the steel is always designed to take the whole of the tension. This point remains to be definitely settled, but Mr. Considere's and Professors Talbot's and Turneaure's tests indicate that there is no dangerous cracking even with high steel until the yield point of the steel is reached. This fact can be positively determined by cutting sections from reinforced concrete beams which have been strained nearly to the elastic limit, and testing them for corrosion by the methods employed by Prof. Charles L. Norton. (See p. 427.) A yield point in steel of 30,000 pounds per square inch corresponds to a stretch of 0.0010 of its length and a yield point of 50,000 to a stretch of 0.00167. (See p. 290.)

A steel with a high modulus of elasticity would be particularly serviceable for reinforced concrete, because the higher the modulus of elasticity of a material, the less is the deformation under any given loading. Unfortunately, however, a high carbon steel has substantially the same modulus of elasticity (30,000,000 lb. per sq. in.) as ordinary merchant steel.

The brittleness feared in high steel is less dangerous in reinforced concrete than in many classes of structural steel work because the concrete protects it from shock, and also because smaller sections of steel are used in concrete beams than in steel beams, and the large and irregular shapes of the latter render them much more sensitive to irregular cooling during the process of their manufacture.

It may be stated, then, if the stretching of high steel when pulled to its allowable working stress is proved not to form dangerous cracks in the concrete, that high carbon steel, say, 0.56% to 0.60% carbon, of the quality used in the United States for making locomotive tires, is always better than mild steel for reinforced concrete provided the steel is well melted and rolled, and is comparatively free from impurities, such as phosphorus. However, a high carbon steel, unless limited by chemical analysis, and made under careful inspection, is in danger of being more brittle than low carbon steel. Its use, therefore, should be limited strictly to work important enough to warrant the ordering of a special steel and the taking of sufficient trouble on the part of the purchaser to insure strict adherence to the specification. Under such circumstances, the use of high steel is attended with much economy. In other words, since manufacturers cannot always be depended upon to exactly follow specifications of this nature, it is necessary that an inspector be sent to the works, or else that the steel be purchased from a reliable dealer who has had it thus carefully tested.

The specifications for first-class steel on page 38 are sufficiently explicit so that steel which comes up to them can be safely used. A steel which can be employed with safety for all the locomotive and car wheels of the country certainly cannot be discarded as unsafe for concrete, provided similar precautions are taken in its purchase.

From Page 68, "Reinforced Concrete," by Buel & Hill:

Grade of Steel.—The quality of steel used in reinforcing concrete should be as carefully specified as for an all-steel structure doing the same duty. Some engineers advocate the use of high steel, on account of its high elastic limit, which recent tests show gives a higher ultimate strength to the beam. The breaking load for beams having the proper amount of reinforcement appears to be at about the elastic limit of the steel. In most cases, and certainly in structure subject to shock or impact, the writer considers it better and more conservative practice to use medium or mild structural steel, except for reinforcement for thermal and shrinkage stresses only, where high steel appears to be preferable.

PROTECTION OF STEEL OR IRON FROM CORROSION.

Most tests which have been conducted of steel imbedded in concrete have resulted in positive proofs of the protection offered by Portland cement concrete, not only from corrosion or rust, but from the most severe fire that is liable to occur. Of course, the steel must be imbedded of sufficient depth in the concrete to obtain these results,—from one to two inches being usually accepted as a safe distance from the surface. While these results are not as readily obtained with a cinder concrete, yet by being thoroughly wet and well mixed, they should be.

Many engineers condemn cinder concrete owing to its extremely porous nature, thereby allowing the moisture and air to penetrate to the steel, which in a comparatively short time will rust it out entirely. In many instances the corrosion of steel in cinder concrete has been attributed to the sulphur contained in the cinders; this, however, is not now accepted as the cause, but is due to the fact that it has not been mixed thoroughly and sufficiently wet.

Cinders often contain Oxide of Iron, and when this is the case, and the mixture is not sufficiently wet to give the steel a thorough coating with cement, it quickly corrodes any steel with which it comes in contact.

The following pages: "Preservation of Iron in Concrete," reprinted from Chapter XII, "Reinforced Concrete," by Buel & Hill, contains some interesting tests on this subject:

Preservation of Iron in Concrete.—It has generally been assumed that iron or steel embedded in concrete does not corrode, and many instances are cited of embedded steel being removed from concrete quite as clear and bright after a long period of exposure to the elements as it was when first embedded. It should be noted, however, that an occasional instance is cited to show that under certain circumstances metal embedded in concrete will corrode. As the durability of concrete-steel requires that the steel shall be permanently protected from corrosion, this question is an important one and it has received consideration from a number of experts. The commonly accepted theory accounting for the protection from rust of iron embedded in concrete has been recently stated by Prof. Spencer B. Newberry as follows:

The rusting of iron consists in oxidation of the metal to the condition of hydrated oxide. It does not take place at ordinary temperatures in dry air or in moist air free from carbonic oxide. The combined action of moisture and carbonic acid is necessary. Ferrous carbonate is first formed; this is at once oxidized to ferric oxide and the liberated carbon dioxide acts on a fresh portion of metal. Once started the corrosion proceeds rapidly, perhaps on account of galvanic action between the oxide and the metal. Water holding carbonic acid in solution soon, if free from oxygen, acts as an acid and rapidly attacks iron. In lime-water or soda solution the metal remains bright. The action of cement in preventing rust is now apparent. Portland cement contains about 63 per cent. lime. By the action of water it is

converted into a crystalline mass of hydrated calcium silicate and calcium hydrate. In hardening it rapidly absorbs carbonic acid and becomes coated on the surface with a film of carbonate, cement mortar thus acting as an efficient protector of iron and captures and imprisons every carbonic-acid molecule that threatens to attack the metal. The action is, therefore, not due to the exclusion of the air, and even though the concrete be porous, and not in contact with the metal at all points, it will still filter out and neutralize the acid and prevent its corrosive effect.

The use of cement washes and plasters for the specific purpose of protecting iron and steel from rust is quite common and has extended over a long period of time. Cement paint is largely used by the railway companies of France to protect their metal bridges from corrosion. Two coats of liquid cement and sand are applied with leather brushes. After investigation and careful tests the engineers of the Boston Subway adopted Portland-cement paint for the protection of the steel beams of that structure. Iron spirit-tanks for European distilleries are universally painted on the inside with Portland-cement paint to prevent corrosion. In the United States it is a frequent practice to coat the inside of steel salt-pans, sulphate digesters, etc., with cement plaster to prevent corrosion. Regarding the damage from corrosion by the sulphur in the cinders of cinder concrete Prof. Newberry expresses himself as follows:

The fear has sometimes been expressed that cinder concrete would prove injurious to iron on account of the sulphur contained in the cinders. The amount of this sulphur is, however, extremely small. Not finding any definite figures in this point, I determined the sulphur contained in an average sample of cinders from Pittsburg coal. The coal in its run state contains a rather high percentage of sulphur, about 1.5 per cent. The cinders proved to contain only 0.61 per cent. sulphur. This amount is quite insignificant, and even if all oxidized to sulphuric acid it would at once be taken up and neutralized in concrete by the cement present, and would by no possibility attack the iron.

In connection with this statement it may be noted that in the demolition in 1903 of a tall steel-frame building in New York City, which was built in 1898 and had practically all of its framework except the columns embedded in cinder concrete, the steel removed showed practically no rust which could be considered as having developed after the metal was embedded.

Tests of a reliable character, made to determine the efficiency of concrete in protecting embedded metal from corrosion, are comparatively few. The most important ones which have been published are those of Mr. Breuillie of France and those of Prof. Charles L. Norton of Boston, Mass. Mr. Breuillie's tests were extended in character and the conclusions drawn from them by the experimenter were: (1) That the cement attacked the iron; (2) that water dissolved the composition which formed at the contact of the two materials; (3) that the adhesion of the steel to the cement disappeared when water passed through the concrete for a certain time; (4) that the weight of the iron salts which adhered to the steel and the normal adhesion between

the steel and the concrete increased with time; (5) in all cases the action of the cement on the iron prevented rust and removed the rust from metal which had ben allowed to corrode before being embedded.

The tests conducted by Prof. Charles L. Norton of the Massachusetts Institute of Technology, Boston, Mass., were of a somewhat different character from those of Mr. Breuillie. Briquettes or blocks were made of neat cement; of 1 part cement and 3 parts sand; of 1 part cement and 5 parts broken stone, and of 1 part cement and 7 parts cinders. Portland cement was used, and was tested chemically and physically and found good. The cinders when washed down with a hose-stream and dried tested alkaline, and analysis revealed very small amounts of sulphur. In each block there was embedded a ¼-in. rod, a piece of soft sheet steel 6×1×232 in., and a 6×1-in. strip of expanded metal. These blocks were exposed as follows: one-quarter of them in sealed chests containing an atmosphere of steam, air, and carbon dioxide; one-quarter in a similar chest with an atmosphere of air and carbon dioxide; one-quarter in a chest with an atmosphere and steam, and one-quarter on a table in the open air of the testing-room. At the end of three weeks the blocks were carefully cut open, and the steel examined and compared with specimens which had lain unprotected in the corresponding chests and in the open air.

The results of the examinations were as follows: The unprotected specimens consisted of rather more rust than steel. The specimens embedded in neat cement were perfectly protected. Of the remaining specimens hardly one had escaped serious corrosion. The location of the rust-spot was invariably coincident with either a rod in the concrete or a badly rusted cinder. In the more porous mixtures the steel was spotted with alternate bright and badly rusted areas, each clearly defined. In both the solid and the porous cinder concrete many rust-spots were found, except where the concrete had been mixed very wet, in which case the watery cement had coated nearly the whole of the steel, like a paint, and protected it. The following are Prof. Norton's conclusions from his tests:

(1) Neat Portland cement, even in thin layers, is an effective preventative of rusting.

(2) Concrete, to be effective in preventing rusting, must be dense and without voids and cracks. It should be mixed quite wet when applied to the metal.

(3) The corrosion found in cinder concrete is mainly due to the iron oxide, or rust, in the cinders and not to the sulphur.

(4) Cinder concrete, if free from voids and well rammed when wet, is about as effective as stone concrete in protecting steel.

(5) It is of the utmost importance that the steel be clean when bedded in concrete. Scraping, pickling, a sand-blast, and lime should be used, if necessary, to have the metal clean when built into a wall.

At first sight the conflicting testimony which has been quoted appears to have but little solid ground upon which the practicing engineer can base a decision as to the probable damage from rust of

iron or steel embedded in concrete. A brief analysis will show, however, that this is not actually the case. In the first place there are many instances where steel embedded in concrete has shown no signs of rust upon removal. None of the evidence presented disputes this fact. Secondly, steel removed from concrete which contained cracks or voids has in many instances shown rust, always at the points where the cracks and voids were located. None of the evidence presented disputes this fact. Thirdly, the theory that the concrete covering filters out and renders innocuous the corrosive elements so completely as to protect the steel even where it is not in contact with the concrete is disputed by the results of Prof. Norton's tests. Fourthly, Prof. Norton's tests show that where the concrete is so closely in contact with the steel as to completely cover it with cement there is no corrosion. This fact is not disputed by any of the other evidence. Fifthly, Prof. Norton's tests show that wet concrete mixtures more certainly insure the close contact of the steel and concrete at all points than do dry mixtures. This fact is not disputed by any of the other evidence. Sixthly, Prof. Norton's contention that the steel should be perfectly cleaned before it is bedded in concrete is controverted by the tests of Mr. Breuille, which show that bedding in concrete will remove the rust from previously corroded steel. Seventhly, all the evidence presented indicates that the sulphur content of the cinders is not a serious element of danger in cinder concrete and that, other conditions being the same, cinder concrete and stone concrete are about equally efficient in preventing the rusting of embedded steel. The useful conclusion which the practicing engineer can draw from all this is that, so far as danger from subsequent rusting is concerned, he can confidently embed steel or iron reinforcement in either cinder or stone concrete if he secures a close contact between the concrete and steel at all points, and if no cracks develop in the concrete to expose the metal to attack.

FIRE PROTECTION.

The value of concrete as a fireproofing is apparently unquestionable, not alone from laboratory experiments, etc., but from fires which have actually occurred in buildings where this material has been employed. (See Fire Test Report on page 127.)

The following from Page 431 of Taylor & Thompson's volume, Concrete Plain & Reinforced, contains some very interesting results of both tests and actual fires:

Numerous experimental tests* have been made showing the value of concrete as a fire-resisting material, but the best proof of its ability to resist the heat of a severe fire—such as is liable to occur in an office or factory building— lies in the fact that concrete has actually withstood very severe fires more successfully than have terra-cotta and various other so-called fireproof materials.

The reinforced concrete factory of the Pacific Coast Borax Co. at Bayonne, N. J., passed through a severe fire in 1902. Still more recently, in 1904, occurred the conflagration at Baltimore in which many building materials utterly failed.

Such practical tests, further confirmed by numerous experiments with test buildings of reinforced concrete, have proved that while in a severe fire, where the temperature ranges from 1600° to 2000° Fahr., the surface of the concrete may be injured to a depth of from ½ to ¾ inch, the body of the concrete is unaffected, so that the only repairs required consist of a coating of plaster, and even this only in rare instances.

Tests upon small briquettes of cement placed in a furnace indicate that the strength of cement is destroyed by a heat reaching a dull, red color,† but as stated below, in an actual fire, the injured material protects the rest of the concrete so that the danger is theoretical rather than real.

Fire in Borax Factory. The fire in the 4-story reinforced concrete factory of the Pacific Coast Borax Company,‡ built entirely of concrete except the roof, utterly destroyed the contents of the building, the roof, and the interior framework, but the walls and floors remained intact except in one place where an 18-ton tank fell through the plank roof and cracked some of the floor beams, and in one place on the outside of the wall where the surface of the concrete was slightly affected. The fire was so hot that brass and iron castings were melted to junk. A small annex, built of steel posts and girders, was completely wrecked, and the metal bent and twisted into a tangled mass.

Baltimore Fire. The effect of the fire upon the concrete in various buildings located in the center of the burned districts of Baltimore

*See References, Chapter XXIX.
†Digest of Physical Tests, Vol. I, p. 217
‡See p. 463.

is best appreciated by an examination of the reports of experts upon the fire. Capt. John S. Sewell, in his report to the Chief of Engineers, U. S. A.,* in referring to the fire in one of the buildings built with reinforced concrete columns, beams, and arches, writes:

It was surrounded by non-fireproof buildings, and was subjected to an extremely severe test, probably involving as high temperature as any that existed anywhere. The concrete was made with broken granite as an aggregate. The arches of the roof and the ceiling of the upper story were cracked along the crown, but in my judgment very slight repairs would have restored any strength lost here. Cutting out a small section — say an inch wide — and caulking it full of good strong cement mortar would have sufficed. The exposed corners of columns and girders were cracked and spalled, showing a tendency to round off to a curve of about 3 in. radius. In the upper stories, where the heat was intense, the concrete was calcined to a depth of from ¼ to ¾ inch, but it showed no tendency to spall, except at exposed corners. On wide, flat surfaces, the calcined material was not more than ¼-inch thick, and showed no disposition to come off. In the lower stories, the concrete was absolutely unimpaired, though the contents of the building were all burned out. In my judgment, the entire concrete structure could have been repaired for not over 20% to 25% of its original cost. On March 10, I witnessed a loading test of this structure. One bay of the second floor, with a beam in the center, was loaded with nearly 300 pounds per sq. ft. superimposed, without a sign of distress, and with a deflection not exceeding ⅛-inch. The floor was designed for a total working load of 150 pounds per sq. ft. The sections next to the front and rear walls were cantilevers, and one of these was loaded with 150 pounds per sq. ft. superimposed, without any sign of distress, or undue deflection.

Captain Sewell concludes as a result of the examination of this and other buildings containing reinforced concrete construction:

As the material is calcined and damaged to some extent by heat, enough surplus material should be provided to permit of a loss of say ¾-inch all over exposed surfaces, if the structure is to be exposed to fire; moreover, all exposed corners should be rounded to a radius of about 3 inches. This latter precaution would add much to the resistance of all materials used in masonry—whether bricks, stone, concrete or terra-cotta—if they are to be exposed to fire.

Concrete Versus Terra-Cotta. Prof. Norton, in his report on the Baltimore fire to the Insurance Engineering Experiment Station,* says:

Where concrete floor arches and concrete-steel construction received the full force of the fire it appears to have stood well, distinctly better than the terra-cotta. The reasons I belive are these; First, because the concrete and steel expand at sensibly the same rate, and hence when heated do not subject one another to stress, but terra-cotta usually expands about twice as fast with increase in temperature as steel, and hence the partitions and floor arches soon become too large to be contained by the steel members which under ordinary temperature properly enclose them. Under this condition the partition must buckle and the segmental arches must lift and break the bonds, crushing at the same time the lower surface member of the tiles.

**Engineering News*, March 24, 1904, p. 276.
**Engineering News*, June 2, 1904, p. 529.

When brick or terra-cotta are heated no chemical action occurs, but when concrete is carried up to about 1 000° Fahr. its surface becomes decomposed, dehydration occurs, and water is driven off. This process takes a relatively great amount of heat. It would take about as much heat to drive the water out of this outer quarter-inch of the concrete partition as it would to raise that quarter-inch to 1 000° Fahr. Now a second action begins. After dehydration the concrete is much improved as a non-conductor, and yet through this layer of non-conducting material must pass all the heat to dehydrate and raise the temperature of the layers below, a process which cannot proceed with great speed.

Cinder Versus Stone Concrete. Prof. Norton compares the action of stone and cinder concrete in the Baltimore fire as follows:

Little difference in the action of the fire on stone concrete and cinder concrete could be noted, and as I have earlier pointed out, the burning of the bits of coal in poor cinder concrete is often balanced by the splitting of the stones in the stone concrete. I have never been able to see that in the long run either stood fire better or worse than the other. However, owing to its density the stone concrete takes longer to heat through.

Further experiments are required to determine the relative durability under extreme heat of concrete made with different kinds of broken stone. It seems probable, from the composition of the rock, that hard trap or gravel may be preferable to limestone, slate, or conglomerate as fire-resisting material.

Thickness of Concrete Required to Protect Metal from Fire. The conclusion reached by Prof Norton† from tests upon concrete arches is that two inches of good concrete gives perfect assurance of safety in case of fire, even if the steel to be protected is in the form of I-beams. Rods of small dimensions can be more effectively coated, and it appears evident from the various tests and from practical experience in severe fires that 1½ inches of concrete around steel rods is sufficient protection. The Pacific Borax Company's fire and other similar tests indicate that in slabs of reinforced concrete, ½ inch to ¾ inch affords ample protection. Secondary members, such as cross girders, or slabs of long span, should have a thickness of concrete outside of the steel varying from ¾ inch to 1½ inch. Although in slabs protected by only ½ inch of concrete, the latter may be softened by an extreme fire, and the metal exposed when it is struck by the stream from a hose, the metal in the majority of cases would still remain practically uninjured, and it is questionable economy to put an excess of material where there is so little probability of its being needed, and where a failure would merely produce local damage.

THEORY OF FIRE PROTECTION.

Mr. Spencer B. Newberry, in an address delivered before the Associated Expanded Metal Companies, Feb. 20, 1902,* gives the following explanation of the fire-proof qualities of Portland cement concrete:

†*Insurance Engineering*, Dec., 1901, p. 483.
**Cement*, May, 1902, p. 95.

The two principal sources from which cement concrete derives its capacity to resist fire and prevent its transference to steel are its *combined water and porosity*. Portland cement takes up in hardening a variable amount of water, depending on surrounding conditions. In a dense briquette of neat cement the combined water may reach 12%. A mixture of cement with three parts sand will take up water to the amount of about 18% of the cement contained. This water is chemically combined, and not given off at the boiling point. On heating, a part of the water goes off at about 500° Fahr., but the dehydration is not complete until 900° Fahr. is reached. This vaporization of water absorbs heat, and keeps the mass for a long time at comparatively low temperature. A steel beam or column embedded in concrete is thus cooled by the volatilization of water in the surrounding cement. The principle is the same as in the use of crystalized alum in the casings of fireproof safes; natural hydraulic cement is largely used in safes for the same purpose.

The porosity of concrete also offers great resistance to the passage of heat. Air is a poor conductor, and it is well known that an air space is a most efficient protection against conduction. Porous substances, such as asbestos, mineral wool, etc, etc., are always used as heat-insulating material. For the same reason cinder concrete, being highly porous, is a much better non-conductor than a dense concrete made of sand and gravel or stone, and has the added advantage of lightness. In a fire the outside of the concrete may reach a high temperature, but the heat only slowly and imperfectly penetrates the mass, and reaches the steel so gradually that it is carried off by the metal as fast as it is supplied.

MODULUS OR COEFFICIENT OF ELASTICITY.

Results of testing concrete for its Modulus of Elasticity for the same mixtures or proportions vary greatly. This is probably due to the exactness necessary in measuring the deformation of concrete. The Modulus of Elasticity of steel varies from 28,000,000 lbs to 31,000,000 lbs. per square inch; 29,000,000 or 30,000,000 being the values usually accepted for steel. Those for concrete, of course, vary with the proportion or the mixtures.

The following "Modulus of Elasticity," reprinted from Taylor & Thompson's, "Concrete Plain & Reinforced," Page 285 suggests the values for the Modulus of Elasticity of concrete:

Modulus of Elasticity. The modulus of elasticity of steel varies from 28,000,000 pounds per square inch to 31,000,000 pounds per square inch; 30,000,000 is customarily taken as an average value, and is the value which we have adopted.

The modulus of elasticity of concrete, a very important factor in reinforced concrete design, is considered in the preceding chapter, page 265. As there stated, it varies with the materials of which it is composed and with the proportions of these materials, also with the method of mixing and placing the concrete.

As tentative values for use in reinforced design, the authors suggest the following moduli for concrete mixed of the wet consistency usually employed in beams:

	Proportions	Modulus of Elasticity lbs. per sq. in.
Broken Stone or Gravel Concretes	1 : 1½ : 3	4 000 000
	1 : 2 : 4	3 000 000
	1 : 2½ : 5	2 500 000
	1 : 3 : 6	2 000 000
	1 : 5 : 8	1 500 000
Cinder Concrete................	1 : 2 : 5	850 000

It is probable that eventually these values will be found too low for dense, well-graded mixtures, which are gradually replacing those proportioned by rule of thumb methods. The authors have found a modulus of about 4 000 000 in 12-inch concrete cubes mixed 1: 2 1-3: 4 2-3, the crushing strength of which was about 5 000 pounds per square inch at the end of two months.

The higher the modulus of elasticity of the concrete, the lower should be the percentage of steel and the greater the depth of the beam for symmetrical design, that is, maintaining fixed relations of pull in steel to pressure in concrete.

From tests of Prof. W. Kendrick Hatt* the modulus of elasticity in tension appears to be of similar value to the compressive modulus. Earlier experimenters concluded that the modulus in tension is lower than in compression. A knowledge of the tensile modulus is, however,

*Journal Association Engineering Societies, June, 1904, p. 323.

of less consequence than the other because the tensile resistance of concrete is not usually considered.

It is probable that there is an increase in the modulus of elasticity of concrete with age, but experiments by the author indicate that this is very slight.

Recent tests,* contrary to former ideas, indicate that under different loadings there may be slight change in the modulus of elasticity of a given concrete until near to its crushing strength. This fact is of importance in fixing the distribution of stresses in the beam.

Elongation or Stretch in Concrete. The question of "Elongation or Stretch in Concrete," is dealt within the succeeding paragraph, reprinted from Page 287 of Taylor & Thompson's volume.

According to tests of Prof. Turneaure, already mentioned, concrete under a pull, as in the lower portion of a beam, will usually stretch 0.0001 to 0.0002 of its length, that is, 0.01% to 0.02%, before showing minute cracks or "water-marks." Cracks become readily noticeable at a stretching varying, in different specimens, from 0.0003 to 0.0010 of their length. The concrete in a reinforced beam stretches similarly to the concrete in a plain beam except that in the latter the beam breaks when the limit of stretch is reached, while if reinforced, the pull is borne partly by the steel and partly by the concrete, and they both stretch together up to the point that cracks so minute at first as to be almost invisible occur in the concrete.

The action of the reinforced concrete is shown in the deflection curve in Fig. 89. The inclination of this curve changes at about the same load that is required to break a similar beam or plain concrete.

The diagram shows a typical result of Prof. Talbot's tests of the deformation of the concrete and the deformation of the steel, the deflection of the beam, and the various measured positions of the neutral axis during flexure. Among other conclusions, Prof. Talbot draws the following:

1. The composite structure acts as a true combination of steel and concrete in flexure during the first or preliminary stage, and this stage lasts until the steel is stressed to, say 3,000 pounds per square inch, and the lower surface of the concrete is elongated about of its length.

2. During the second or readjustment stage there is a marked change in distribution of stresses, the neutral axis rises, the concrete loses part of its tensional valve, and tensile stresses formerly taken by the concrete are transferred to the steel. During this stage minute cracks probably exist, quite well distributed, and not easily detected.

3. In the third or straight-line stage the neutral axis remains nearly stationary in position and the concrete gradually loses more of its tensional value. Visible cracks appear and gradually grow larger, though no change in the character of the load-deformation diagram results. It would seem probable that at these cracks the stress in the steel is more than is indicated by the average deformation for the full gage length.

*See Discussion on Concrete, by Sanford E. Thompson, International Eng. Congress, St. Louis, 1904.

Prof. Talbot states that at the load when the curve changes character,—which in the beam shown in the diagram is about 8 000 pounds total load, — there are probably invisible cracks in the lower portion of the beam. This change in direction of the curve, indicating a suddenly increased load upon the steel, is strong proof of the loss in tensional resistance of the concrete. Prof Turneaure, moreover, in his experiments, at loads somewhat beyond the point of change in direction, actually discovered these minute cracks. He tested his beams upside down, that is, the load was applied upward, and the minute cracks or water-marks were shown by hair lines on the wet surface of the concrete. Prof Turneaure* says:

It has been found that by testing the beams when somewhat moist, a crack is made visible when exceedingly small, it appearing first as a narrow, wet streak perhaps ⅛-inch wide and a little later as a dark hair-like crack. It was not necessary to search for the lines with a microscope as under these conditions they were readily found.

That the wet streak, called a "water-mark" hereafter, shows the presence of an actual crack was demonstrated last year by sawing out a strip of the concrete containing such a water-mark; the strip fell apart at the water-mark.

In the plain concrete no water-marks or cracks were observed before rupture. Comparing the observed and calculated elongations of the reinforced concrete with those for the plain concrete at rupture it will be seen that the initial cracking in the former occurs at an elongation practically the same as in the latter.

The significance of these minute cracks is an open question. It has been supposed that concrete reinforced by steel will elongate about ten times as much before rupture as will plain concrete. These experiments show very clearly that rupture begins at about the same elongation in both cases. In the plain concrete total failure ensues at once; in the reinforced concrete rupture occurs gradually, and many small cracks may develop so that the total elongation at final rupture will be greater than in the plain concrete. In other words, the steel develops the full extensibility of a non-homogeneous material that otherwise would have an extension corresponding to the weakest section.

*Proceedings American Society for Testing Materials, 1904

BONDING OLD AND NEW CONCRETE.

Too much attention cannot be paid by constructors or contractors to the bonding of old and new concrete. In most instances, sufficient care is not given to this in construction. The following from Page 376 of "Concrete Plain & Reinforced," Taylor & Thompson, should be carefully noted:

In a foundation or other structure where the strain is chiefly compressive, the surface of the concrete laid on the previous day should be cleaned and wet, but no other precaution is necessary. Joints in walls or in other locations liable to tensile stress are coated with mortar, which should be richer in cement than the mortar in the concrete, proportions 1: 2 being commonly used.

Some engineers spread the cement dry upon the wetted surface of the old concrete, while others make it into a mortar; the latter method is necessary in many cases to seal the joints between the top of the old concrete and the bottom of the raised forms.

The adhesive strength of cement or concrete is much less than its cohesive strength, hence in building thin walls for a tank or other work which must be water-tight, the only sure method is to lay the structure as a monolith, that is, without joints. If the wall is to withstand water pressure and cannot be built as a monolith, both horizontal and vertical joints must be first thoroughly cleaned of all dirt and "laitance" or powdery scum, wet, and then covered with a very thin layer of either neat cement or 1: 1 mortar, according to the nature of the work. As an added precaution, one or more square or V-shaped sticks of timber, say 4 or 6 inches on an edge, may be imbedded in the surface, or placed vertically at the end of a section, of the last mass of concrete laid each day. In some instances large stones have been partially imbedded in the mass at night for doweling the new work next day.

In the New York Subway, work was commenced with no provision for bonding horizontal layers, but it was soon found that more or less seepage occurred, and in one case where a large arch was torn down the division line between two days' work was distinctly seen. Accordingly, at the end of each day's concreting a tongue-and-grooved joint was formed by a piece of timber 4 inches square partly imbedded in the top layer. This was removed before resuming work.

Roughening the surface after ramming or before placing the new layer will aid in bonding the old and new concrete.

EFFECT OF FREEZING.

The much discussed subject of the effect of freezing or frost upon Portland cement concrete seems to still be a question in the minds of many. This is possibly due, in some cases, to the confusion of Natural and Portland cements. Most natural cements are completely ruined by freezing, while Portland cements seem uninjured.

Numerous tests and investigations have been made in recent years, both in practical work and laboratories; the results being that the only permanent injury is to the surface, which may scale off in frozen before setting, and that the hardening and setting is retarded.

In practice the materials are often heated which causes the cement to set more quickly; or a limited amount of salt may be added to the water, with apparently no injury to the concrete.

The following reprinted from Chapter XIX, Taylor & Thompson's volume, "Concrete Plain & Reinforced," is most interesting:

Numerous experimental tests have been made, chiefly in the United States, where the effect of frost is a more serious question than in England, France, or Germany, to determine the effect of freezing temperatures upon hydraulic cements. Although the conclusions of different experimenters are not in perfect accord, it is the generally accepted belief, corroborated by tests under the most practical conditions and by the appearance of concrete and mortar in masonry construction, that the ultimate effect of freezing upon Portland cement concrete and mortar is to produce only surface injury.

In their practice and research the authors have never discovered a case, either in laboratory work or in practical construction, where Portland cement concrete or mortar laid with proper care has suffered more than surface disintegration from the action of frost. They do not wish to imply however, that it is always expedient to lay Portland cement masonry in freezing weather, for the expense of laying is increased, and it is much more difficult to satisfactorily mix and place the materials. Mortar for brick and stone masonry freezes in the tubs and in the joints, while in laying concrete the surface freezes unless measures are taken to prevent it, and any dirt or "laitance" which rises to the surface of wet mixtures is hard to remove. It is a well-known fact that a thin crust about $\frac{1}{16}$ inch thick is apt to scale off from granolithic or concrete pavements which have frozen, leaving a rough instead of a troweled wearing surface, and the effect upon concrete walls is often similar. It may be stated as a general rule that concrete work should, if possible, be avoided in freezing weather, although if circumstances warrant the added expense, with proper precaution and careful inspection mass concrete may be laid with Portland cement at almost any temperature.

Most Natural cements, on the contrary, are seriously injured by frost especially by alternate freezing and thawing, and while occasional cases are on record, especially in heavy stone masonry in

which the weighted joints have thawed slowly, where Natural cement mortar has been laid in freezing weather without serious results, numerous examples might be cited where even after several years the concrete or mortar was but slightly better than sand and gravel. Mr. Thompson has observed this result in Natural cement mortar laid during the comparatively warm winter of North Carolina on days when the temperature was considerably above freezing at the time of laying, and also in the cold climate of Maine where the mortar froze as it left the trowel and did not thaw until spring.

The settlement of the masonry when thawing is often a serious characteristic of Natural cements. Stone masonry walls laid in freezing weather in Natural cement mortar may settle as much as ½ inch in the height of a window jamb.

Experiments upon Natural cement mortars have not positively confirmed the judgment reached by nearly all engineers experienced in construction in freezing weather. Occasional tests are recorded in which such mortars, especially when subjected to a uniformly cold temperature and then suddenly thawed, have attained full strength, but these are insufficient to warrant the use of any except Portland cements when frost is likely to occur before the mortar is thoroughly dry.

The prevention of injury from frost in certain cements may be due, at least in part, to the internal heat produced when setting. In the interior of a large mass, some cements, especially high grade Portlands, attain a high temperature. (See p. 130.)

CLASSIFICATION OF CEMENTS.

Chapter V. reprinted by permission from "Concrete Plain and Reinforced," by Taylor & Thompson:

From an engineering standpoint, limes and cements may be classified as

- Portland cement.
- Natural cement.
- Puzzolan cement.
- Hydraulic lime.
- Common lime.

Typical analyses of each are presented in the following table. The composition of Natural cement, even different samples of the same brand, is so extremely variable that their analyses cannot be regarded as characteristics of locality.

Typical Analysis of Cements.

	PORTLAND CEMENT		NATURAL CEMENT							COMMON LIME	
			AMERICAN		ENGL'H	FRENCH					
	Lehigh Valley[1] (mixed rock)	Western[2] (marl and clay)	Eastern Rosendale[3]	Western Louisville[3]	Roman[4]	Vassy[5]	Grappiers[6]	Puzzolan Cement[7]	Hydraulic Lime (Le Tiel)[8]	Lime[9]	Magnes'n Lime[10]
Silica SiO_2	21.31	21.93	18.38	20.42	25.48	22.60	26.5	28.95	21.70	1.03	1.12
Alumina Al_2O_3	6.89	5.98	15.20	4.76	10.30	8.90	2.5	11.40	3.19	1.27	0.68
Iron Oxide Fe_2O_3	2.53	2.35		3.40	7.44	5.30	1.5	0.54	0.66		
Calcium Oxide CaO	62.89	62.92	35.84	46.64	44.54	52.69	63.0	50.29	60.70	97.02	58.51
Magnesian Oxide MgO	2.64	1.10	14.02	12.00	2.92	1.15	1.0	2.96	0.85	0.68	39.69
Sulphuric Acid SO_3	1.34	1 54	0.93	2.57	2.61	3.25	0.5	1.37	0.60		
Loss on Ignition	1.39	2.91	3.73	6.75	3.68	6.11	5.0	3.39	12.20		
Other constituents	0.75		11.46	3.74	1.46			0.30	0.10		

[1] W. F. Hillebrand, Society of Chemical Industry, 1902, Vol. XXI.
[2] W. F. Hillebrand, Journal American Chemical Society, 1903, 25, 1180.
[3] Clifford Richardson, *Brickbuilder*, 1897, p. 229.
[4] Stanger & Blount, Mineral Industry, Vol. V., p. 69.
[5] Candlot, Ciments et Chaux Hydrauliques, 1898, p. 174.
[6] Le Chatelier, Annales des Mines, September and October, 1893, p. 36.
[7] Report of the Board of U. S. Army Engineers on Steel Portland Cement, 1900, p. 52.
[8] Candlot, Ciments et Chaux Hydrauliques, 1898, p. 24.
[9] Rockland Rockport Lime Co.
[10] Western Lime and Cement Co.

PORTLAND CEMENT.

Portland cement is defined by Mr. Edwin C. Eckel of the U. S. Geological Survey as follows: "By the term Portland cement is to be understood the material obtained by finely pulverizing clinker produced by burning to semi-fusion an intimate artificial mixture of finely ground calcareous and argillaceous materials, this mixture consisting approximately of 3 parts of lime carbonate to 1 part of silica, alumina and iron oxide."

The definition is often further limited by specifying that the finished product must contain at least 1.7 times as much lime, by weight, as of silica, alumina, and iron oxide together.

The only surely distinguishing test of Portland cement is its chemical analysis and its specific gravity. (See pp. 64 and 65.) In the field it may often be recognized by its cold bluish gray color (see p. 113), although the color of Puzzolan and of some Natural cement is so similar that this is by no means a positive indication.

The term *Natural Portland Cement* arose from the discovery in Boulogne-sur-Mer, France, as early as 1846, of a natural rock of suitable composition for Portland cement. A similar discovery in Pennsylvania gave rise to the same term in America, but the manufacturers soon found it necessary to add to the cement rock a small percentage of purer limestone. Since the chemical composition of Portland cement, as defined above, is substantially uniform regardless of the materials from which it is made, in the United States the terms "natural" and "artificial" are meaningless.

In France, cements intermediate between Roman and Portland are called "natural Portlands."*

Sand Cement. Sand or silica cement is a mechanical mixture of Portland cement with a pure, clean sand very finely ground together in a tube mill or similar machine. For the best grades the proportions of cement to sand are 1:1, although as lean a mixture as 1:6 has been made to compete with Natural cements. The coarser particles in any Portland cement have little cementitious value, hence if a portion of the cement is replaced by inert matter and the whole ground extremely fine, its advocates maintain that the product is scarcely inferior to the unadulterated article. As made in the United States, the mixture is ground so fine that 95 per cent of it will pass through a sieve having 200 meshes to the linear inch, and all of the 5 per cent residuum is said to be sand. In other words, all of the cement passes a No 200 sieve.

*Candlot's Ciments et Chaux Hydrauliques, 1898, p. 164.

NATURAL CEMENT.

Natural cement is "made by calcining natural rock at a heat below incipient fusion, and grinding the product to powder."* Natural cement contains a larger proportion of clay than hydraulic lime, and is consequently more strongly hydraulic. Its composition is extremely variable on account of the difference in the rock used in manufacture.

Natural cements in the United States in numerous instances bear the names of localities where first manufactured. For example, Rosendale cement, a term heard in New York and New England more frequently than Natural cement, was originally manufactured in Rosendale. Ulster County, N. Y. Louisville cement first came from Louisville, Ky. The James River, Milwaukee, Utica, and Akron are other Natural cements named for localities.

The United States produces a few brands of "Improved Natural Hydraulic Cement," intermediate in quality between Natural and Portland, by mixing inferior Portland cement with Natural cement clinker.

In England the best known Natural cement is called Roman cement. Occasionally one hears the term Parker's cement, so called from the name of the discoverer in England.

LE CHATELIER'S CLASSIFICATION OF NATURAL CEMENTS.

In France there are several classes of natural cement. Mr. H. Le Chatelier† classifies Natural Cements as those obtained "by the heating of limestone less rich in lime than the limestone for hydraulic lime. They may be divided into three classes:

"Quick-setting cements, such as Vassy and Roman (Ciments a prise rapide, Vassy, romain);
"Slow-setting cements (Ciments a prise demi-lente);
"Grappiers cements (Ciments de grappiers).

"**Vassy Cements** are obtained by the heating of limestone containing much clay, at a very low temperature, just sufficient to decarbonate the lime. They are characterized by a very rapid set, followed afterwards by an extremely slow hardening, much slower than that of Portland cements."

"They differ from Portland cements by containing a much higher percentage of sulphuric acid, which appears to be one of their essential elements, and a much lower percentage of lime.

"**Slow Setting Cements**, by the high temperature of calcination, approach Portland cements, but the natural limestones never possess the homogeneity of artificial mixtures, so that it is impossible to avoid in these cements the presence of a large quantity of free lime." The composition of these products varies from that of the Vassy cements to that of the real Portlands.

*Professional Papers, No. 28, U. S. Army Engineers, p. 33.
†Procedes d'Essai des Materiaux Hydrauliques, Annales des Mines, 1893.

"**Grappiers Cements** are obtained by the grinding of particles which have escaped disintegration in the manufacture of hydraulic limes. These grappiers are a mixture of four distinct materials, two of which, completely inert, are unburned limestone and the clinkers formed by contact with the siliceous walls of furnaces, and two of which, strongly hydraulic, are unslacked lime and true slow-setting cement. It is necessary that the latter should predominate in the grappiers for their grinding to give a useful product. The grappier of cement is obtained regularly only by the heating of a limestone but slightly aluminous and containing about three equivalents of carbonate of lime for one of silica; its production necessitates a heating at high temperature.

"These grappiers cements are even more apt to contain free lime than the Natural cements of slow set which are obtained by the heating of limestone containing much more alumina. Because of their constitution, also, the grappiers cements may vary greatly in composition since they are produced by the grinding of a mixture of grains of cement and of various inert materials. The cement grains have very nearly the composition of tricalcium silicate (SiO_2 3 CaO)."

PUZZOLAN OR SLAG CEMENT.

Puzzolan cement is the product resulting from mixing and grinding together in definite proportions slaked lime and granulated blast furnace slag or natural puzzolanic matter (such as puzzolan, santorin earth, or trass obtained from volcanic tufa).

The ancient Roman cements belonged to the class of Puzzolans. They were made by mechanically mixing slaked lime with natural puzzolana formed from the fusion of natural rock found in the volcanic regions of Italy. In Germany, trass, a volcanic product related to Puzzolan, has been used with lime in the manufacture of cements.

Blast furnace slag is essentially an artificial puzzolana, formed by the combustion in a blast furnace, and the puzzolan or slag cements of the United States are ground mixtures of granulated blast furnace slag, of special composition, and slaked lime.

A Board of Engineers officers, U. S. A., presented in 1901 the following conclusion, * based, undoubtedly, on the exhaustive studies upon the subject made by a previous Board† having the same chairman, Major W. L. Marshall:

This term (slag or Puzzolan cement) is applied to cement made by intimately mixing by grinding together granulated blast-furnace slag of a certain quality and slaked lime, without calcination subsequent to the mixing. This is the only cement of the Puzzolan class to be found in our markets (often branded Portland), and as true Portland cement is now made having slag for its hydraulic base, the term "slag cement" should be dropped and the generic term Puzzolan be used in advertisements and specifications for such cements.

*Professional Papers, No. 28, p. 28.
†Report of the Board of U. S. Army Engineers on Steel Portland Cement, 1900, p. 52.

Puzzolan cement made from slag is characterized physically by its light lilac color; the absence of grit attending fine grinding and the extreme subdivision of its slaked lime element; its low specific gravity (2.6 to 2.8) compared with Portland (3 to 3.5); and by the intense bluish green color in the fresh fracture after long submersion in water, due to the presence of sulphides, which color fades after exposure to dry air.

The oxidation of sulphides in dry air is destructive of Puzzolan cement mortars and concretes so exposed. Puzzolan is usually very finely ground, and when not treated with soda sets more slowly than Portland. It stands storage well, but cements treated with soda to quicken setting become again very slow setting, from the carbonization of the soda (as well as the lime) element after long storage.

Puzzolan cement properly made contains no free or anhydrous lime, does not warp or swell, but is liable to fail from cracking and shrinkage (at the surface only) in dry air.

Mortars and concretes made from Puzzolan approximate in tensile strength similar mixtures of Portland cement, but their resistance to crushing is less, the ratio of crushing to tensile strength being about 6 to 7 to 1 for Puzzolan, and 9 to 11 to 1 for Portland. On account of its extreme fine grinding Puzzolan often gives nearly as great tensile strength in 3 to 1 mixtures as neat.

Puzzolan permanently assimilates but little water compared with Portland, its lime being already hydrated. It should be used in comparatively dry mixtures well rammed, but while requiring little water for chemical reactions, it requires for permanency in the air constant or continuous moisture.

Puzzolanic material has been suggested by Dr. Michaelis, of Germany, and Mr. R. Feret, of France (see Chapter XVIII), as a valuable addition to Portland cement designed for use in sea water.

HYDRAULIC LIME.

The hydraulic properties of a lime,—its ability to harden under water,—are due to the presence of clay, or, more correctly, to the silica contained in the clay. Hydraulic lime is still used to quite an extent in Europe, especially in France, as a substitute for hydraulic cement. The celebrated lime-of-Teil of France is a hydraulic lime.

Mr. Edwin C. Eckel states * that "theoretically the proper composition for a hydraulic limestone should be calcium carbonate 86.8%, silica 13.2%. The hydraulic limestones in actual use, however, usually carry a much higher silica percentage, reaching at times to 25%; while alumina and iron are commonly present in quantities which may be as high as 6%. The lime content of the limestones commonly used varies from 55% to 65%."

Although the chemical composition of hydraulic lime is similar to Portland cement, its specific gravity is much lower, lying between 2.5 and 2.8.†

**American Geologist*, March, 1902, p. 52.
†Candlot's Ciments et Chaux Hydraulique, 1898, p. 26.

In the manufacture of hydraulic lime the limestone of the required composition is burned, generally in continuous kilns, and then sufficient water is added to slake the free lime produced so as to form a powder without crushing.

COMMON LIME.

The commercial lime of the United States is "quicklime," which is chiefly calcium oxide (CaO).

Lime is now manufactured by a continuous process. Limestone of a rather soft texture, so as to be as free as possible from silica, iron and alumina, is charged into the top of the kiln which may be, say, 40 ft. high by 10 ft. in diameter. The fuel is introduced into combustion chambers near the foot of the shaft, and the finished product is drawn out from time to time through another opening in the bottom of the shaft. The temperature of calcination may range from 1400° Fahr. (760° Cent.) to, at times, 2,000° Fahr. (1,090° Cent.). The product (see analysis, p. 47), in ordinary lime of the best quality, is nearly pure calcium oxide (CaO). Upon the addition of water the lime slakes, forming calcium hydrate (CaH_2O_2), and, with the continued addition of water increases in bulk to twice or three times the original loose and dry volume of the lump lime as measured in the cask. In this plastic condition it is termed by plasterers "putty" or "paste."

The setting of lime mortar is the result of three distinct processes which, however, may all go on more or less simultaneously. First, it dries out and becomes firm. Second, during this operation, the calcic hydrate, which is in solution in the water of which the mortar is made, crystalizes and binds the mass together. Hydrate of lime is soluble in 831 parts of water at 78° Fahr; in 759 parts at 32° and in 1136 parts at 140°. Third, as the per cent. of water in the mortar is reduced and reaches five per cent., carbonic acid begins to be absorbed from the atmosphere. If the mortar contains more than five per cent. this absorption does not go on. While the mortar contains as much as 0.7 per cent. the absorption continues. The resulting carbonate probably unites with the hydrate of lime to form a sub-carbonate, which causes the mortar to attain a harder set, and this may finally be converted to carbonate. The mere drying out of mortar, our tests have shown, is sufficient to enable it to resist the pressure of masonry, while further hardening furnishes the necessary bond.*

Magnesian Limes evolve less heat when slaking, expand less, and set more rapidly than pure lime. A typical analysis is given an page 47.

Hydrated Lime is a powdered slaked lime (calcium hydrate). It is manufactured by treating finely ground common lump lime with water of a certain temperature, and then cooling and screening it through a very fine screen.

* The authors are indebted to Mr. Clifford Richardson for this paragraph.

FINISHING SURFACES OF REINFORCED CONCRETE.

Objections are often heard as to the unsightly appearance of concrete buildings when finished. With a little care concrete structures may be made as beautiful to the eye as buildings built of any other material.

The following chapter, XVII, reprinted from Buel & Hill's volume, "Reinforced Concrete," will be found most interesting on this subject, dealing with the numerous finishes which may be applied at very little cost.

CHAPTER XVII.—FACING AND FINISHING EXPOSED CONCRETE SURFACES.

The difficulty of securing an even-grained surface of uniform color on concrete work is one of the most annoying which builders of such work have to overcome. Concrete work is subject to various sorts of surface imperfection, but the two most common imperfections are roughness or irregular surface texture and variability of color or discoloration. Either of these imperfections is capable of disfiguring an otherwise sightly structure, and the task of avoiding them is one which warrants serious attention from those undertaking work in reinforced concrete. Unfortunately practice has not settled upon a solution of the problem, hence its consideration here is rather a record of experience than a set of instructions which can be followed with the certainty that successful results will ensue.

Causes of Roughness and Discoloration.—There are several conditions which may result in a concrete surface of uneven texture and with mechanical roughnesses, such as projections, bulges, ridges, pits, bubble-holes, and scales. One of these is imperfections in the molds. The use of rough lagging of uneven thickness and with open cracks and allowing the forms to become distorted and warped are certain to leave their impress upon the plastic concrete in the form of ridges, tongues, and bulges. Failure to pack the concrete filling tightly and evenly against the mold will result in rough places. Lack of homogeneity in the concrete is another prolific cause of variation in the surface texture of concrete work. This lack of homogeneity may result from failure to mix the concrete materials thoroughly and evenly in the first place, or the segregation of the coarse and fine parts of the mixture during its deposition and ramming into place. In both cases the result is a material of alternate coarse and fine texture. Dirt or cement adhering to the molds will leave pits in the concrete surface, and the pulling away of the concrete in spots when it adheres to the molds when they are removed will cause similar roughness.

Variations in the color of concrete surfaces probably result from a variety of causes. Some of these are obvious and others are difficult to determine with any exactness. Roughness or uneven surface texture is a common cause of variation in color, since the alternate rough

and smooth parts weather differently and collect and hold dirt and soot in different degrees. Another cause of variation in color is the use of different cements in adjacent parts of the surface work. No two cements are of exactly the same shade of color, and the concrete made of them partakes of this variation. In a similar manner sand of different shades of color or of different degrees of cleanliness will cause a cloudy and streaky appearance in concrete . Dirt adhering to the molds will frequently stain the adjacent concrete surface.

Even when the smoothness of the surface is satisfactory, however, and when there is no criticism possible as to the kind and quality of the aggregates, their deposition and the cleanliness with which the work is done, concrete surfaces frequently vary in color and have a cloudy light and dark appearance. In many cases there seems to be good reason for attributing this to the leaching out of lime, compounds and their deposition in the form of an efflorescence on the concrete surface. The extent of this efflorescence varies; at times the deposit is so thin as merely to give a lighter shade to the places where it appears, but it will often form an encrustation of considerable body and thickness which may be readily scraped off as a white or yellowish-white powder. The nature of this discoloration and the preventive and remedial treatments which have been practiced in its cure are discussed more fully in a succeeding paragraph.

Construction of Forms.—Very slight imperfections in the face of the forms against which the concrete is molded are sufficient to leave an unsightly impression on the plastic mixture when it hardens. Even the grain of smoothly dressed timber will show on the surface of concrete which has been deposited with a mortar facing. It is very difficult to construct forms so that they will not leave slight impressions of this character, and it is generally better not to attempt the task in any but exceptional instances. In these a straight-grained, smoothly dressed timber, with its pores filled with soap or paraffine well rubbed in, or a rougher timber covered with sheet metal, can be used. Generally speaking, all has been done that is practicable so far as the forms are concerned when the face-lagging is kept true to surface and has close-fitting joints. Grain-marks and similar minor impressions of the forms can usually be eliminated by rubbing the surface or floating it with grout, at less cost, than by attempting to perfect the molds beyond a reasonable measure. In fact many engineers experienced in concrete work prefer not to attempt to secure particularly perfect finish in the forms, but to dress the entire surface by some style of tooling or rubbing process after the forms have been removed. The most apparent imperfection in concrete surfaces is usually the joint-marks of the lagging-boards. These may be due either to slight differences in the thickness of adjoining boards or to open joints. The remedy for the first cause is obvious, but it is not so easy to insure

smooth, tight joints and keep them smooth and tight when the boards swell from the moisture absorbed from the wet concrete. One of the most successful forms of joint is that shown by the sketch Fig. 78. In this construction the wedge edge presses into the edge of the adjoining board without distorting or bulging the lagging. Pointing the joints with hard soap or putty, packing them with oakum and covering them with pasted strips of cloth, are other means which have been practiced for preventing joint-marks on the concrete. A method of eliminating grain-marks, which was used with success in constructing piers of the Frazer River Bridge in British Columbia, consisted in covering the tightly laid matched lagging with gloss oil and then blowing sand into the oil with hand-bellows.

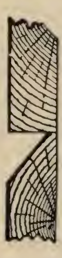

FIG. 78.

Mortar or Grout Facing.—One of the most frequently employed means for securing a smooth surface finish on concrete is to use a mortar or grout facing. This facing differs from plastering in being laid on as the concrete is deposited, thus forming a single piece with it. The thickness of mortar facing employed in practice varies from ½ in. to 3 ins., but the usual practice is to make it 1 in. or 1½ ins. thick. A facing as thick as 3 ins. is rather unnecessary waste of mortar, while one which is much less than 1 in. thick is likely to be pierced by the stones in the concrete unless great care is taken in ramming the concrete filling behind the mortar facing. A mortar or grout facing shows the impress of small roughnesses on the mold more readily than does concrete, and particular care is necessary to secure a smooth surface in the mold when the mortar facing is adopted. The composition of the facing mortar is usually specified as 1 part of Portland cement to 2 or 3 parts of sand. These ingredients are mixed rather wet, since the paste must completely fill the facing-mold, but care must be had not to have so thin a paste that the stones from the concrete behind will be pushed through it during the subsequent filling and ramming.

The following method of placing mortar facing is practiced by the Illinois Central R. R. and has gained wide adoption during the last few years. A sheet-iron plate 6 or 8 ins. wide and about 6 ft. long has riveted across it on one side 1½-in. angles spaced about 2 ft. apart. One edge of this plate is provided with handles. This device is employed as a mold for the facing and is operated in the following manner: The plate is set up against the face of the form with its angle-ribs close against the timber and its handles upward. In this position of the plate there is between it and the form an open slot 1½ ins. wide. This slot is filled with mortar which is tamped thoroughly, and immediately afterwards the concrete backing is deposited behind the plate. When this has been done the plate is withdrawn by the handles and the backing and facing are rammed together to a close bond. The mortar facing is mixed in small batches as it is needed. and no delay is permitted in placing the concrete backing, the

essential principle and purpose of the method being to secure as nearly as is possible the simultaneous construction of the backing of concrete and its facing of mortar.

Fig. 79 shows an excellent form of surface mold of the type just described. By varying the size of the angle-ribs any desired thickness of facing can be constructed, and the flare of the top edge facilitates the placing of the mortar, which is usually done with shovels. In lieu of a steel plate, use is sometimes made of a board provided with furring-strips on one side. This is a more unwieldly device than the one illustrated, and it is objectionable because of the large crevice left upon withdrawal into which the mortar facing is likely to slough and which is less easily closed and bonded by the final ramming. In constructing mortar facing with either iron or board molds perfect success is secured only at the expense of great care. The mortar must be mixed in small batches and only as needed, and it must be thoroughly rammed and churned into the facing-mold. The concrete backing must be deposited behind the mold without delay and firmly rammed against it, and finally the ramming together of the facing and backing must be thorough.

The following method of applying grout facing was employed with success in constructing the Atlantic Avenue subway for the Long Island R. R. in Brooklyn, N. Y. The concrete was deposited in 6-in. or 8-in. layers, and after ramming the concrete at the face was pushed back from the form about 1 in. with an ordinary gardener's spade and a thick grout of 1 part cement and 2 parts sand was poured into the space. The forms used were tongued and grooved yellow pine painted with paraffine paint. In this work a good surface was invariably secured when the men did their work faithfully, but any carelessness on their part evidenced itself in a rough spot when the forms were removed. As an indication of the susceptibility of mortar facing in taking impressions from the forms, it may be noted that even with the dressed and paraffined lagging the grain of the wood was shown perfectly on the mortar facing.

Finishing Mortar Facing.—When mortar or grout facing is employed as described in the preceding paragraphs the slightest imperfections in the grain of smoothly dressed wood is clearly impressed on the plastic material. There will also be occasional rough spots, pittings, or bubble-holes even with the most careful construction. To get rid of these some method of surface finishing must be resorted to. A number of methods have been practiced. In recent concrete culvert work on the New York Central & Hudson River R. R. an excellent surface finish was obtained by the following procedure: The forms of 2-in. dressed and matched pine, after being put in place, were painted with a coat of thin soft soap, then as the layers of concrete were brought up the face was drawn back with a square-pointed shovel, the edges of which had been hammered flat. Mortar in the proportion of 1 part cement to 2 parts sand, mixed rather wet, was then poured in along the form and the layer rammed against it. Hard

soap was used to fill openings left by points of the lagging. When the forms were removed and while the concrete was yet "green," the surface was carefully rubbed with a circular motion, with pieces of white firebrick or briquettes of 1 cement to 1 sand, made in molds about the size of a building brick, handles being pressed in while soft. The surface was then dampened and painted with a coat of grout of 1 cement to 1 sifted sand, and this was closely followed by a final rubbing with a circular movement, using a wooden float. All edges were rounded with a Crafts edger, or with wood fillet, and the coping joints were struck with Crafts jointer.

In the specifications for concrete presented by the special committee of the Engineering and Maintenance of Way Association the following requirement for finishing was adopted:

After the forms are removed, any small cavities or openings in the concrete shall be neatly filled with mortar if necessary. Any ridges due to cracks or joints in the lumber shall be rubbed down; the entire face shall then be washed with a thin grout of the consistency of whitewash, mixed in the proportion of 1 part of cement to 2 parts of sand. The wash should be applied with a brush.

In the extensive concrete construction of the Aurora, Elgin & Chicago R. R. the exposed surfaces were all finished according to the following specifications:

All walls when finished must present a smooth, uniform surface of cement mortar, and all disfigurements must be effaced, and if there are any open, porous places, they must be neatly filled with mortar of 1 cement and 2 sand, well rubbed in, which finishing must be done immediately upon the removal of the forms. Compensation for all labor and material required in such finishing, including the mortar facing when required, with the finishing of bridge seats and other parts, is included in the price per cubic yard for concrete work.

Mr. Edwin Thacher, in his general specifications for concrete-steel requires the following surface finish:

For plain flat surfaces, the concrete may be rammed directly against the molds, and, after the molds have been removed, all exposed surfaces shall be floated to a smooth finish with semi-liquid mortar, composed of 1 part cement and 2 parts of fine, sharp sand, care being taken that no body of mortar is left on the face, sufficient only being used to fill the pores and give a smooth finish.

A very effective finish is obtained by etching the mortar facing with acid. The method consists of using a facing mortar composed of Portland cement and finely crushed stone, the kind of stone depending upon the appearance desired. Thus any shade of red or gray granite, sandstone, etc., can be obtained, and special effects can be obtained by the use of sand, pigments, etc., in the mixture. This mortar is composed of about 1 part Portland cement to 2 or 3 parts of the finely crushed stone. The exposed surfaces are then treated by chemical or mechanical means to remove the cement matrix at the face, leaving

the granular particles of stone partly exposed. In general this is done by washing the surface with a weak acid solution, then with clean water, and finally with an alkiline solution to neutralize any effects of the acid. In the finished work it is difficult to detect that the material is not natural stone, except by close inspection. The stone is crushed to pass through a sieve of 10 to 30 meshes per square inch according to the character of finish desired, and enough water is used to make a soft plastic mixture.

Plastering.—Plastering as a method of finishing concrete surfaces deserve mention for the purpose alone of calling a warning against its adoption. It is practically impossible to apply mortar in thin layers to a concrete surface and make it adhere for any length of time, and when it once begins to scale off the result is a surface many times worse in looks than the unfinished concrete that it was intended to render more sightly.

Pebble Dash Facing.—An effective surface finish for certain classes of concrete work can be secured by using large rounded pebbles in place of the usual aggregate for the surface layer of concrete, and then, while the concrete is soft, removing the mortar between the pebbles by wire brushing until approximately half the pebbles are exposed. The following specification for this style of facing was employed in constructing a small concrete road-bridge in the National Park at Washington, D. C.:

The concrete, which will be in the exterior faces of the bridge and the parapet walls for a thickness of 18 ins., will be made of gravel and rounded stone varying in the concrete below the belting course between 1½ and 2 ins. in their smallest diameters. This gravel will be mixed in the concrete as aggregate instead of broken stone. The mixture will consist of 1 part Portland cement, 2 parts sand, and 5 parts of aggregate. The parapet walls will be made in a similar manner, with the aggregate composed of gravel not exceeding 1 in. in its smallest diameter. When the forms are removed the cement and sand must be brushed from around the face of the gravel with steel brushes, leaving approximately half of the gravel exposed.

In this work it was found by test that at the age of 12 hours the concrete was not sufficiently set to hold the pebbles from being torn out by the brushing, and that at the age of 36 hours it was too hard to permit the brushing, to remove a sufficient depth of mortar without undue labor. At 24 hours' age the brushing proved most successful.

Tooled Surfaces.—A method of finishing concrete surfaces which is preferred by many experienced engineers is to dress the concrete after it has hardened by means of hammers or pointed chisels. The process is exactly analogous to stone dressing, and any of the forms of finish employed for cut stone can be employed equally well for concrete. In connection with tooled surfaces it is common to mold the concrete to represent ashlar masonry by means of horizontal and vertical V-shaped depressions formed as shown by Fig. 80. This style of finish has been extensively employed by Mr. E. L. Ransome, who gives the following directions for securing it: In imitating rough-

dressed work the mold is removed from the concrete while it is yet tender, and with small light picks the face is picked over with great rapidity, an ordinary workman finishing about 1,000 square feet per day. For imitations of finer-tooled work the concrete should be left to harden longer before being spalled or cut, and the work should be done with a chisel. Most natural stone and especially granite makes excellent material for the face, but ordinary gravel will do. Whatever is used, let it be uniform in color and of even grade. When a very fine and close imitation of a natural stone is required take the same stone, crush it and mix it with cement colored to correspond. The finer the stone is crushed the nearer the resemblance will be upon close inspection; but for fine work it is generally sufficient to reduce the stone to the size of buckshot or fine gravel.

Masonry Facing.—A facing of masonry is often employed on reinforced-concrete arch bridges, and is a very satisfactory solution of the problem of surface finish for such structures. Masonry facing may be of any style of stonework which is used for true masonry arches, and coursed ashlar, random rubble, and boulder masonry facings have all been employed. Exactly the same care should be exercised in selecting stones and laying them up into arch ring and wall, cornice and parapet, as if the structure were entirely of masonry. Beyond this the most important feature to be observed is close bonding of the masonry facing to the concrete backing. To insure this there should be a liberal use of stretchers reaching well into the backing, and these can be supplemented with metal cramps to the advantage of the work in many instances. For facing the arch ring the stones should be cut to true voussoir shape, and laid quite as perfectly as if they were a part of a true voussoir arch ring. The soffit of the arch ring is not stone-faced. In place of stone a brick facing may be employed.

The following specifications for stone and brick facing, which were prepared by Mr. Edwin Thacher, M. Am. Soc. C. E., to control work conducted by him, give a fair idea of the requirements of high-class work of this character:

Stone Facing.—If stone facing is used, the ring stones, cornices, and faces of spandrels, piers, and abutments shall be of an approved quality of stone. The stone must be of a compact texture, free from loose seams, flaws, discolorations, or imperfections of any kind, and of such a character as will stand the action of the weather. The spandrel-walls will be backed with concrete, or rubble masonry, to the thickness required. The stone facing shall in all cases be securely bonded or clamped to the backing. All stone shall be rock-faced with the exception of cornices and string courses, which shall be sawed or bush-hammered. The ring stones shall be dressed to true radial lines, and laid in Portland cement mortar, with ¼ in. joints. All other stones shall be dressed to true beds and vertical joints. No joint shall exceed ½ in. in thickness and shall be laid to break joints at least 9 ins. with the course below. All joints shall be cleaned, wet, and neatly pointed. The faces of the walls shall be laid in true lines, and to the dimensions given on plans, and the corners shall have a

chisel draft 1 in. wide carried up to the springing lines of the arch, or string course. All cornices, moldings, capitals, keystones, brackets, etc., shall be built into the work in the proper positions and shall be of the forms and dimensions shown on plans.

Brick Facing with Concrete Trimmings.—The arch rings, cornices, string courses, and quoins shall be concrete-faced as described above, the arch rings and quoins being marked and beveled to represent masonry. The piers, abutments, and spandrels shall be faced with vitrified brick, as shown on plans. The brick facing shall be plain below the springing lines of the arches, and rock-faced above these lines. All rock-faced brick shall be chipped by hand from true pitch lines. All brick-facing shall be bonded as shown on plans, at least one-fifth of the face of the wall being headers. The brick must be of the best quality of hard-burned paving brick, and must stand all tests as to durability and fitness required by the engineer in charge. The bricks must be regular in shape and practically uniform in size and color They shall be free from lime and other impurities; shall be free from checks or fire cracks, and as nearly uniform in every respect as possible; shall be burned so as to secure the maximum hardness; so annealed as to reach the ultimate degree of toughness ;and be thoroughly vitrified so as to make a homogeneous mass.

The backing shall be carried up simultaneously with the face work, and be thoroughly bonded with it.

The use of boulder facing will ordinarily be limited to structures of special character, and its success will depend very largely upon the care with which the stones are selected, their size, and their arrangement in the structure. In constructing a boulder-faced concrete arch at Washington, D. C., the following requirements were specified for the facing:

The term boulder here is meant to cover loose rock, which shall be hard, durable, and of a quality to be approved by the engineer, whose edges have become weathered or water-worn, or both, and are more or less rounded. It is the intention to obtain a decidedly rustic effect on the facing, and to that end extreme care must be taken in the selection of the stones, and only mechanics who show an aptitude for this class of work shall be employed. No tool marks or fresh fractures will be allowed on the showing faces.

The boulder face of such stone shall project at least 2 ins. beyond the neat lines of the bridge, and this projection shall not exceed 15 ins., nor shall it be greater than one-half the least horizontal dimension of the stone. All joints shall be scraped and brushed clear of mortar to the depth indicated by the engineer. The mortar shall consist of 1 part Portland cement and 2 parts sand. The backs of all boulders shall be plastered with a layer of mortar as specified, at least ¼ in. thick, immediately before ramming the concrete against them.

The arch-stones shall have a depth of between 3 and 4 ft., a width of not less than 18 ins., nor more than 36 ins.; all dimensions to be measured exclusive of the projections beyond the neat lines. The joints shall be dressed so as not to exceed 1½ ins. at any point for at least two-thirds their depth and two-thirds their length, and as much more as the stones will admit. Each arch-stone shall be cramped to the adjacent steel girder by means of a wrought-iron cramp made from ¾ × ⅜-in. bar, the cramps to reach at least 2 ins. into each boulder, to be well cemented into them, and securely cramped to the top of the girder. The outside girders shall be cramped to the adjacent girders by 10 wrought-iron cramps made from ¾ × ⅜-in. bar (in construction we used ¾-in., as it bent cold without fracture).

No dressing will be required on the stones used in abutments, spandrels, and wing walls of the work, but only well-shaped boulders, laid on their broadest bed, will be allowed. Dressing will be permitted on such stones as cannot be properly bedded without it. The parapet walls will be a continuation of the spandrel and wing walls. The boulder stone must reach entirely through the wall.

Cast Concrete Slab Veneer.—In constructing the arch bridge at Soissons, France, which is described on p. 244, the faces of the arch-ribs and the spandrel facing were formed of slabs of concrete-steel molded separately and set in place like stone veneer with the remainder of the concrete forming a backing. An essentially similar construction was employed in Chicago, Ill., in 1902, in constructing a number of recreation buildings in one of the city parks. In the last example mentioned the slabs were cast face down in wooden molds; the mode of procedure being as follows:

A layer of mortar composed of 1 part cement and 2 or 3 parts of finely crushed stone was first placed in the bottom of the mold to a depth of from ½ in. to 1 in.; on this bed of mortar a 1-2-4, concrete, with ½ to ¾ in. stone, was placed to the thickness desired and carefully rammed. After hardening, the blocks were removed from the molds and set aside to season until they were placed in the structure.

The construction of the slab veneer for the Soissons Bridge was as follows: For molding the arch-rib facing a smooth level platform or pavement of concrete was constructed on an adjacent level piece of ground. This molding platform was large enough to permit the arch-ribs to be delineated to full size on its surface. To prepare the mold the platform was covered smoothly with gunny cloth held down by battens, which also served to outline the extrados and intrados of the arch-rib. Radical strips of wood were then placed to divide the mold into voussoir-like sections. A thin bed of mortar was placed on the bottom of the mold and on this was laid four reinforcing-bars, one near and parallel to each edge of the voussoir being molded, so as to intersect at the corners. Under these bars at several points wire stirrups were looped with their fine ends projecting upward. The metal was then covered with a rich concrete of fine stone laid on the mortar-bed and compacted so that the total thickness was about 2 ins. When hardened, the product of the mold was a set of voussoir-shaped slabs with smooth faces and edges and a rough back with a number of projecting wires. In construction these facing slabs were set in place with mortar joints and backed with concrete. For the spandrel-wall facing the slabs were cast in rectangular molds in exactly the same manner. The engineers of the Soissons Bridge remark that the use of this cast concrete veneer enabled a considerable reduction of expense for forms and assured a surface finish of pleasing appearance.

Moldings and Ornamental Shapes.—The finishing of concrete structures in many instances comprehends the construction of moldings and ornamental shapes for cornices, corbels, medallions, key-stones, and other architectural parts. These may either be molded in place

by suitable construction of the stationary forms or they may be cast separately in portable molds and set in place in the structure as would be cut stone. Panels of simple form or plain cornice moldings can usually be molded in place without great trouble and expense, but in constructing corbels, complicated moldings, balusters, etc., particularly where one pattern is duplicated a number of times, time and expense will usually be saved by casting them separately or in sections, and afterwards erecting the separate pieces in the structure.

The casting of ornamental shapes in concrete may be accomplished either in sand-molds or in rigid molds of wood, metal, or plaster of Paris. Some very handsome work has been recently performed by sand-molding. The mode of procedure followed in making concrete castings in sand varies somewhat in practice, but it is substantially as follows: A pattern of the shape to be cast is first made in wood and to the exact size required, since no allowance for shrinkage is necessary. The pattern is then molded in sand in flasks exactly as is done in casting iron. The mixture used usually consists of cement and finely crushed stone of about the consistency of cream, and this is poured into the mold by means of a funnel and T pipe. The excess water in the mixture soaks into the sand and serves to keep the casting moist during setting. Generally the casting is left in the mold for three or four days, and is then removed and the projecting fins, if any, are cut off. The cast stone may be used immediately in the work, but it is preferable to let it season and harden for a fortnight or more before using. The product of these sand-molds has an unusually attractive surface texture. Sand-molding is particularly advantageous when balusters, corbels, medallions, and intricate moldings have to be cast, but for plain cornices and facing slabs it is generally as cheap and convenient to use wooden forms.

Efflorescence.—The leaching out of certain lime compounds and their deposition on the surfaces of concrete work are quite frequently the cause of the uneven color of such surfaces. In relation to this source of discoloration Mr. Clifford Richardson, Director of the New York Testing Laboratory, says:

> It is primarily due to variations in the amount of water in the mortar of which the cement is composed. It will be readily understood that when any excess of water is used, segregation of the coarse and fine particles will take place, with a resulting difference in color. When a large amount of water is used the concrete is more porous and the very considerable percentage of free lime liberated from the Portland cement in the course of setting is more readily brought to the surface at such point. . . . The amount of water in a concrete, the face of which is to be exposed, should be neither too small nor too large, but such a concrete should certainly not be dry or the exposed face will be honeycombed. . . . Where the greatest care is used as to the amount of water added to the mortar and to prevent its loss, and where separation of the mortar from the broken stone is carefully avoided in depositing the concrete and in ramming it, the exposed surface, after the removal of the molds, is fairly uniform in color. . . . A more uniform color will always be obtained when some puzzolanic material is ground in with the cement such as slag or tross. This

hydrated silicious material combines with the lime which has been liberated and prevents it washing out on the surface. . . . Exactness in the amount of water used in the concrete, when the elimination of the stain caused by the free lime is considered desirable, the addition of some substance containing silica in an active form are the two steps to be taken to produce a concrete surface which should present a uniform color and a pleasing appearance.

The measures whose adoption are recommended in the quotation just made are designed to prevent the occurrence of efflorescence by adopting certain precautions in the materials and workmanship of the original construction. Their adoption, however, if it gives the success that Mr. Richardson anticipates, is obviously the way to get at the root of the trouble, but such action involves a degree of skill and watchfulness in constructing concrete work which is difficult of attainment under ordinary conditions of engineering construction, and which if attained will add materially to the cost of construction. They have the further objection that a special mixture of cement is required about which our information is not entirely certain. In default of preventive measures, which recommend themselves to general use, the engineer who encounters the trouble of efflorescence must overcome it by remedial measures. There are a number of these available. The most practical ones are the washing of the discolored surfaces by solution, which will remove the incrustation, or the removal of the original surface by dressing it down with hammers or tooling of some sort.

The manner of dressing down concrete surfaces to eliminate surface imperfections is discussed in a previous paragraph. The following account of the method of cleaning a concrete-steel bridge at Washington, D. C., gives instructive data as to this mode of procedure: The bridge in question had a mortar facing and after this was completed a heavy rain caused the entire north facade to become discolored by efflorescence. This discoloration was not uniform, but in streaks and blotches of a white color, which after weathering a short time turned into a dirty yellow. To clean the bridge trial was first made of water and wire brushes, but after a little work this method was considered impracticable owing chiefly to its cost, which was estimated at $2.40 per square yard. Washes of dilute hydrochloric acid, of dilute acetic acid, and of dilute oxalic acid were then tried in conjunction with ordinary scrubbing-brushes. The hydrochloric-acid wash proved the best, and the acetic-acid wash came next in efficiency. The wash finally adopted consisted of a solution of 1 part hydrochloric

acid and 5 parts water. This was applied vigorously with scrubbing-brushes, water being constantly played on the work with a hose to prevent the penetration of the acid. One house-cleaner and five laborers were employed on the work, which cost 60 cents per square yard. This high cost was due largely to the difficulty of cleaning the balustrades; it was estimated that the cost of cleaning the spandrel- and wing-walls did not exceed 20 cents per square yard. The cleaning was thoroughly satisfactory. Some of the flour removed by the brushes was analyzed and found to be silicate of lime.

TABLES OF QUANTITIES OF MATERIALS.

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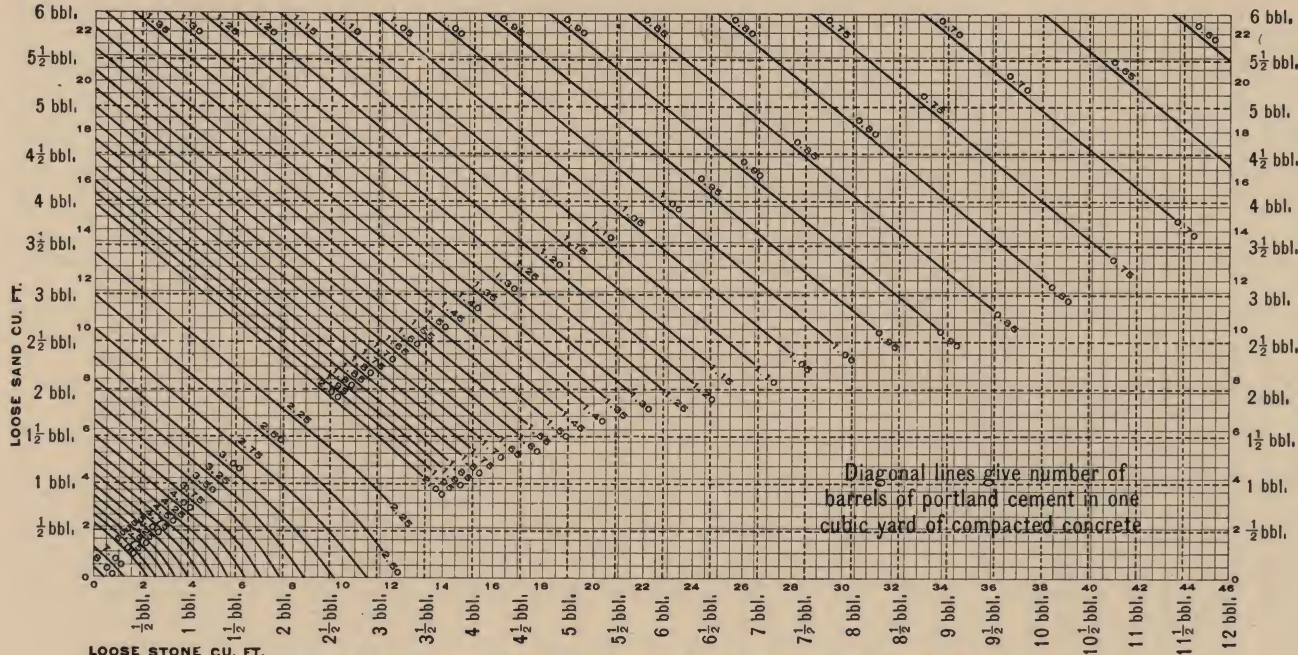

BARRELS OF PORTLAND CEMENT FOR ONE CUBIC YARD OF CONCRETE IN VARIOUS PROPORTIONS.

BASED ON STONE HAVING 45% VOIDS

FIG. 77. (See important rules below, also examples on page 226, and formula (7) on page 224.)

1. **Dotted cross section lines** represent barrels or parts of sand and stone to one barrel (4 bags) Portland cement weighing 376 lbs. and measuring (assumed) 3.8 cu. ft.

2. **Full cross section lines** represent cubic feet of sand and stone to one barrel (4 bags) Portland cement.

3. **Diagonal lines** represent number of barrels of Portland cement in one cubic yard of compacted concrete.

4. **To find number of cubic yards of sand or stone** per cubic yard of concrete, multiply number of barrels cement, as above, by 0.141 times the number of parts of sand or stone.

5. **To find number of cubic feet of concrete,** in any proportions, made from one barrel of cement, divide 27 by the number of barrels of cement per cubic yard, obtained as above.

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VOLUME OF PLASTIC MORTAR MADE FROM DIFFERENT PROPORTIONS OF CEMENT AND SAND.

QUANTITIES OF MATERIALS PER CUBIC YARD. (SEE P. 227.)

Relative proportions by volume*		Volume of Compacted Plastic Mortar from 1 cu. ft. Cement, Based on Portland Cement weighing			Volume of Compacted Plastic Mortar from 1 bbl. Cement, Based on barrel of			Materials for 1 cu. yd. Compact Plastic Mortar Based on barrel of 3.5 cu. ft.		Materials for 1 cu. yd. Compact Plastic Mortar Based on barrel of **3.8 cu. ft.†**		Materials for 1 cu. yd. Compact Plastic Mortar Based on barrel of 4 cu. ft.	
Cement	Sand	108 lbs. per cu. ft.	**100 lbs. per cu. ft.†**	95 lbs. per cu. ft.	3.5 cu. ft.	**3.8 cu. ft†**	4 cu. ft.	Packed Cement	Loose Sand	**Packed Cement**	**Loose Sand**	Packed Cement	Loose Sand
		cu. ft.	**cu. ft.**	cu. ft.	cu. ft.	**cu. ft.**	cu. ft.	bbl.	cu. yd.	**bbl.**	**cu. yd**	bbl.	cu. yd.
1	0	0.93	0.86	0.80	3.2	3.2	3.2	8.31		8.31		8.31	
1	½	1.12	1.06	1.02	3.9	4.0	4.1	6.92	0.46	6.73	0.47	6.61	0.49
1	1	1.48	1.42	1.38	5.2	5.4	5.5	5.22	0.68	5.01	0.71	4.88	0.72
1	1½	1.84	1.78	1.74	6.4	6.7	7.0	4.20	0.81	4.00	0.84	3.87	0.86
1	2	2.20	2.14	2.11	7.7	8.1	8.4	3.51	0.91	3.32	0.93	3.21	0.95
1	2½	2.56	2.50	2.47	9.0	9.5	9.9	3.01	0.98	2.84	1.00	2.74	1.01
1	3	2.92	2.86	2.83	10.2	10.9	11.3	2.64	1.03	2.48	1.05	2.39	1.06
1	3½	3.28	3.23	3.19	11.5	12.2	12.8	2.35	1.06	2.20	1.08	2.12	1.10
1	4	3.64	3.59	3.55	12.8	13.6	14.2	2.12	1.10	1.98	1.11	1.90	1.13
1	4½	4.01	3.95	3.91	14.0	15.0	15.6	1.92	1.12	1.80	1.14	1.72	1.15
1	5	4.37	4.31	4.28	15.3	16.4	17.1	1.77	1.15	1.65	1.16	1.58	1.17
1	5½	4.73	4.67	4.64	16.6	17.7	18.5	1.63	1.16	1.52	1.18	1.46	1.19
1	6	5.09	5.03	5.00	17.8	19.1	20.0	1.52	1.18	1.41	1.19	1.35	1.20
1	6½	5.45	5.39	5.36	19.1	20.5	21.4	1.41	1.19	1.32	1.21	1.26	1.21
1	7	5.81	5.76	5.72	20.3	21.9	22.9	1.33	1.21	1.23	1.21	1.18	1.22
1	7½	6.18	6.12	6.08	21.6	23.2	24.3	1.25	1.21	1.16	1.22	1.11	1.23
1	8	6.54	6.48	6.44	22.9	24.6	25.8	1.18	1.22	1.10	1.24	1.05	1.24

NOTE.—Variations in the fineness of the sand and the cement, and in consistency of the mortar may affect the values by 10% in either direction.

*Cement as packed by manufacturer, sand loose.

† **Use these columns ordinarily.**

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QUANTITIES OF MATERIALS FOR ONE CUBIC YARD OF RAMMED CONCRETE. BASED ON A BARREL OF 3.5 CUBIC FEET.

(See important foot-notes, also p. 225)

PROPORTIONS BY PARTS			PROPORTIONS BY VOLUMES			Volume of mortar in terms of percentage of volume of stone	PERCENTAGES OF VOIDS IN BROKEN STONE OR GRAVEL: 50%*			45%†			40%‡			30%§			20%§		
Cement	Sand	Stone	Packed Cement	Loose Sand	Loose Stone		Cement	Sand	Stone	Cement	Sand	Stone	Cement	Sand	Stone	Cement	Sand	Stone	Cement	Sand	Stone
			bbl	cu. ft.	cu. ft.	%	bbl.	cu. yd.	cu. yd.	bbl	cu. yd.	cu. yd.	bbl	cu. yd.	cu. yd.	bbl	cu. yd.	cu. yd.	bbl	cu. yd.	cu. yd.
1		1	1		3.5	101	5.25		0.68	5.07		0.66	4.89		0.63	4.51		0.58	4.19		0.54
1		2	1		7.0	54	3.84		1.00	3.64		0.94	3.47		0.90	3.09		0.80	2.80		0.73
1		3	1		10.5	39				2.85		1.11	2.69		1.05	2.35		0.91	2.10		0.82
1		4	1		14.0	31										1.90		0.99	1.69		0.88
1		5	1		17.5	27										1.59		1.03	1.41		0.91
1		6	1		21.0	24										1.37		1.07	1.21		0.94
1		7	1		24.5	21													1.06		0.96
1		8	1		28.0	20													0.94		0.98
1		9	1		31.5	18													0.84		0.98
1		10	1		35.0	17													0.77		1.00
1		11	1		38.5	16													0.70		1.00
1		12	1		42.0	16													0.65		1.01
1	1	1½	1	3.5	5.2	104	3.37	0.44	0.65	3.26	0.42	0.63	3.15	0.41	0.61	2.95	0.38	0.57	2.78	0.36	0.54
1	1	2	1	3.5	7.0	78	3.02	0.39	0.78	2.89	0.38	0.75	2.78	0.36	0.72	2.58	0.33	0.67	2.41	0.31	0.62
1	1	2½	1	3.5	8.7	64	2.73	0.35	0.88	2.60	0.34	0.84	2.49	0.32	0.80	2.29	0.30	0.74	2.12	0.28	0.68
1	1	3	1	3.5	10.5	54	2.49	0.32	0.97	2.37	0.31	0.92	2.25	0.29	0.88	2.06	0.27	0.80	1.90	0.25	0.74
1	1½	2	1	5.2	7.0	95	2.64	0.51	0.68	2.55	0.49	0.66	2.46	0.47	0.64	2.30	0.44	0.60	2.16	0.42	0.56
1	1½	2½	1	5.2	8.7	78	2.42	0.47	0.78	2.32	0.45	0.75	2.23	0.43	0.72	2.07	0.40	0.67	1.93	0.37	0.62
1	1½	3	1	5.2	10.5	65	2.23	0.43	0.87	2.13	0.41	0.83	2.04	0.39	0.79	1.88	0.36	0.73	1.74	0.34	0.68
1	1½	3½	1	5.2	12.2	56	2.07	0.40	0.94	1.97	0.38	0.89	1.88	0.36	0.85	1.72	0.33	0.78	1.59	0.31	0.72
1	1½	4	1	5.2	14.0	50	1.93	0.37	1.00	1.83	0.35	0.95	1.74	0.34	0.90	1.59	0.31	0.82	1.46	0.28	0.76
1	1½	4½	1	5.2	15.7	45	1.81	0.35	1.05	1.71	0.33	0.99	1.62	0.31	0.94	1.47	0.28	0.86	1.35	0.26	0.78
1	1½	5	1	5.2	17.5	41	1.70	0.33	1.10	1.60	0.31	1.04	1.52	0.29	0.99	1.37	0.26	0.89	1.25	0.24	0.81
1	2	3	1	7.0	10.5	77	2.02	0.52	0.79	1.94	0.50	0.75	1.86	0.48	0.72	1.73	0.45	0.67	1.61	0.42	0.63
1	2	3½	1	7.0	12.2	67	1.89	0.49	0.85	1.80	0.47	0.81	1.73	0.45	0.78	1.59	0.41	0.72	1.48	0.38	0.67
1	2	4	1	7.0	14.0	59	1.77	0.46	0.92	1.69	0.44	0.88	1.61	0.42	0.83	1.48	0.38	0.77	1.37	0.35	0.71
1	2	4½	1	7.0	15.7	53	1.67	0.43	0.97	1.58	0.41	0.92	1.51	0.39	0.88	1.38	0.36	0.80	1.27	0.33	0.74
1	2	5	1	7.0	17.5	48	1.57	0.41	1.02	1.49	0.39	0.97	1.42	0.37	0.92	1.29	0.33	0.84	1.18	0.31	0.76
1	2	5½	1	7.0	19.2	44	1.49	0.39	1.06	1.41	0.36	1.00	1.34	0.35	0.95	1.21	0.31	0.86	1.11	0.29	0.79
1	2	6	1	7.0	21.0	41	1.42	0.37	1.10	1.34	0.35	1.04	1.27	0.33	0.99	1.14	0.30	0.89	1.04	0.27	0.81
1	2½	3	1	8.7	10.5	90	1.84	0.59	0.72	1.78	0.57	0.69	1.71	0.55	0.66	1.60	0.52	0.62	1.50	0.48	0.58
1	2½	3½	1	8.7	12.2	78	1.73	0.56	0.78	1.66	0.53	0.75	1.60	0.52	0.72	1.48	0.48	0.67	1.38	0.44	0.62
1	2½	4	1	8.7	14.0	68	1.63	0.52	0.85	1.56	0.50	0.81	1.50	0.48	0.78	1.38	0.44	0.72	1.28	0.41	0.66
1	2½	4½	1	8.7	15.7	61	1.55	0.50	0.90	1.47	0.47	0.86	1.41	0.45	0.82	1.29	0.42	0.75	1.20	0.39	0.70
1	2½	5	1	8.7	17.5	55	1.47	0.47	0.95	1.39	0.45	0.90	1.33	0.43	0.86	1.22	0.39	0.79	1.12	0.36	0.73
1	2½	5½	1	8.7	19.2	51	1.39	0.45	0.99	1.32	0.42	0.94	1.26	0.41	0.90	1.15	0.37	0.82	1.06	0.34	0.75
1	2½	6	1	8.7	21.0	47	1.33	0.43	1.03	1.26	0.41	0.98	1.20	0.39	0.93	1.09	0.35	0.85	1.00	0.32	0.78
1	2½	6½	1	8.7	22.7	44	1.27	0.41	1.07	1.20	0.39	1.01	1.14	0.37	0.96	1.03	0.33	0.87	0.94	0.30	0.79
1	2½	7	1	8.7	24.5	41	1.22	0.39	1.11	1.15	0.37	1.04	1.09	0.35	0.99	0.98	0.32	0.89	0.90	0.29	0.82
1	3	4	1	10.5	14.0	77	1.52	0.59	0.79	1.46	0.57	0.76	1.40	0.54	0.73	1.30	0.50	0.67	1.21	0.47	0.63
1	3	4½	1	10.5	15.7	69	1.44	0.56	0.84	1.38	0.54	0.80	1.32	0.51	0.77	1.22	0.47	0.71	1.13	0.44	0.66
1	3	5	1	10.5	17.5	62	1.37	0.53	0.89	1.31	0.51	0.85	1.25	0.48	0.81	1.15	0.45	0.75	1.07	0.42	0.69
1	3	5½	1	10.5	19.2	57	1.31	0.51	0.93	1.25	0.48	0.89	1.19	0.46	0.85	1.09	0.42	0.78	1.01	0.39	0.72
1	3	6	1	10.5	21.0	53	1.25	0.48	0.97	1.19	0.46	0.93	1.13	0.44	0.88	1.03	0.40	0.80	0.95	0.37	0.74
1	3	6½	1	10.5	22.7	49	1.20	0.47	1.01	1.14	0.44	0.96	1.08	0.42	0.91	0.98	0.38	0.82	0.90	0.35	0.76
1	3	7	1	10.5	24.5	46	1.15	0.45	1.04	1.09	0.42	0.99	1.03	0.40	0.93	0.94	0.36	0.85	0.86	0.33	0.78
1	3	7½	1	10.5	26.2	43	1.11	0.43	1.08	1.05	0.41	1.02	0.99	0.38	0.96	0.90	0.35	0.87	0.82	0.32	0.80
1	3	8	1	10.5	28.0	40	1.06	0.41	1.10	1.01	0.39	1.05	0.95	0.37	0.99	0.86	0.33	0.89	0.78	0.30	0.81
1	4	5	1	14.0	17.5	77	1.22	0.63	0.79	1.17	0.61	0.76	1.12	0.58	0.73	1.04	0.54	0.67	0.97	0.50	0.63
1	4	6	1	14.0	21.0	65	1.12	0.58	0.87	1.07	0.55	0.83	1.02	0.53	0.79	0.94	0.49	0.73	0.87	0.45	0.68
1	4	7	1	14.0	24.5	56	1.04	0.54	0.94	0.99	0.51	0.90	0.94	0.49	0.85	0.86	0.44	0.78	0.80	0.41	0.73
1	4	8	1	14.0	28.0	50	0.97	0.50	1.01	0.92	0.48	0.95	0.87	0.45	0.90	0.80	0.41	0.83	0.73	0.38	0.76
1	4	9	1	14.0	31.5	45	0.91	0.47	1.06	0.86	0.44	1.00	0.81	0.42	0.94	0.74	0.38	0.86	0.68	0.35	0.79
1	4	10	1	14.0	35.0	41	0.85	0.44	1.10	0.81	0.42	1.05	0.76	0.39	0.98	0.69	0.36	0.89	0.63	0.33	0.82
1	5	10	1	17.5	35.0	48	0.79	0.51	1.02	0.75	0.49	0.97	0.71	0.46	0.92	0.65	0.42	0.84	0.59	0.38	0.76
1	6	12	1	21.0	42.0	46	0.67	0.52	1.04	0.63	0.49	0.98	0.60	0.47	0.93	0.54	0.42	0.84	0.50	0.39	0.78

NOTE.—Variations in the fineness of the sand and the compacting of the concrete may affect the quantities by 10% in either direction.

*Use 50% columns for broken stone screened to uniform size.

†Use 45% columns for average conditions and for broken stone with dust screened out.

‡Use 40% columns for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

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QUANTITIES OF MATERIALS FOR ONE CUBIC YARD OF RAMMED CONCRETE. BASED ON A BARREL OF 3.8 CUBIC FEET.

(See important foot-notes, also p. 225.)

Proportions by Parts: Cement	Sand	Stone	Proportions by Volumes: Packed Cement	Loose Sand	Loose Stone	Volume of mortar in terms of percentage of volume of stone	50%*: Cement	Sand	Stone	45%†: Cement	Sand	Stone	40%‡: Cement	Sand	Stone	30%§: Cement	Sand	Stone	20%§: Cement	Sand	Stone
			bbl.	cu. ft.	cu. ft.	%	bbl.	cu. yd.	cu. yd.	bbl.	cu. yd.	cu. yd.	bbl.	cu. yd.	cu. yd	bbl.	cu. yd.	cu yd.	bbl.	cu. yd.	cu. yd.
1		1	1		3.8	94	5.09		0.72	4.90		0.69	4.73		0.67	4.33		0.61	4.02		0.57
1		2	1		7.6	51	3.67		1.03	3.48		0.98	3.30		0.93	2.93		0.82	2.65		0.75
1		3	1		11.4	36				2.69		1.14	2.54		1.07	2.22		0.94	1.98		0.84
1		4	1		15.2	29										1.78		1.00	1.58		0.89
1		5	1		19.0	25										1.49		1.05	1.31		0.92
1		6	1		22.8	22										1.28		1.08	1.12		0.95
1		7	1		26.6	20													0.98		0.97
1		8	1		30.4	19													0.87		0.98
1		9	1		34.2	18													0.78		0.99
1		10	1		38.0	17													0.71		1.00
1		11	1		41.8	16													0.65		1.01
1		12	1		45.5	15													0.60		1.01
1	1	1½	1	3.8	5.7	99	3.19	0.45	0.67	3.08	0.43	0.65	2.97	0.42	0.63	2.78	0.39	0.59	2.62	0.37	0.55
1	1	2	1	3.8	7.6	75	2.85	0.40	0.80	2.73	0.38	0.77	2.62	0.37	0.74	2.43	0.34	0.68	2.26	0.32	0.64
1	1	2½	1	3.8	9.5	61	2.57	0.36	0.90	2.45	0.34	0.86	2.34	0.33	0.82	2.15	0.30	0.76	1.99	0.28	0.70
1	1	3	1	3.8	11.4	51	2.34	0.33	0.99	2.22	0.31	0.94	2.12	0.30	0.90	1.93	0.27	0.82	1.77	0.25	0.75
1	1½	2	1	5.7	7.6	93	2.49	0.53	0.70	2.40	0.51	0.68	2.31	0.49	0.65	2.16	0.46	0.61	2.03	0.43	0.57
1	1½	2½	1	5.7	9.5	76	2.27	0.48	0.80	2.18	0.46	0.77	2.09	0.44	0.74	1.94	0.41	0.68	1.80	0.38	0.63
1	1½	3	1	5.7	11.4	64	2.09	0.44	0.88	2.00	0.42	0.84	1.91	0.40	0.81	1.76	0.37	0.74	1.63	0.34	0.69
1	1½	3½	1	5.7	13.3	55	1.94	0.41	0.96	1.84	0.39	0.91	1.76	0.37	0.87	1.61	0.34	0.79	1.48	0.31	0.73
1	1½	4	1	5.7	15.2	49	1.80	0.38	1.01	1.71	0.36	0.96	1.63	0.34	0.92	1.48	0.31	0.83	1.36	0.29	0.77
1	1½	4½	1	5.7	17.1	44	1.69	0.36	1.07	1.60	0.34	1.01	1.51	0.32	0.96	1.37	0.29	0.87	1.25	0.26	0.79
1	1½	5	1	5.7	19.0	40	1.59	0.34	1.12	1.50	0.32	1.06	1.42	0.30	1.00	1.28	0.27	0.90	1.17	0.25	0.82
1	2	3	1	7.6	11.4	75	1.89	0.53	0.80	1.81	0.51	0.76	1.74	0.49	0.74	1.61	0.45	0.68	1.50	0.42	0.63
1	2	3½	1	7.6	13.3	65	1.76	0.49	0.87	1.68	0.47	0.83	1.61	0.45	0.79	1.48	0.42	0.73	1.38	0.39	0.68
1	2	4	1	7.6	15.2	57	1.65	0.46	0.93	1.57	0.44	0.88	1.50	0.42	0.84	1.38	0.39	0.78	1.27	0.36	0.72
1	2	4½	1	7.6	17.1	51	1.55	0.44	0.98	1.48	0.42	0.94	1.41	0.40	0.89	1.28	0.36	0.81	1.18	0.33	0.75
1	2	5	1	7.6	19.0	47	1.47	0.41	1.03	1.39	0.39	0.98	1.32	0.37	0.93	1.20	0.34	0.84	1.10	0.31	0.77
1	2	5½	1	7.6	20.9	43	1.39	0.39	1.08	1.31	0.37	1.01	1.25	0.35	0.97	1.13	0.32	0.87	1.03	0.29	0.80
1	2	6	1	7.6	22.8	40	1.32	0.37	1.11	1.25	0.35	1.06	1.18	0.33	1.00	1.06	0.30	0.89	0.97	0.27	0.82
1	2½	3	1	9.5	11.4	87	1.72	0.61	0.73	1.66	0.58	0.70	1.60	0.56	0.68	1.49	0.52	0.63	1.40	0.49	0.59
1	2½	3½	1	9.5	13.3	75	1.62	0.57	0.80	1.55	0.55	0.76	1.49	0.52	0.73	1.38	0.49	0.68	1.29	0.45	0.64
1	2½	4	1	9.5	15.2	66	1.52	0.54	0.86	1.46	0.51	0.82	1.40	0.49	0.79	1.29	0.45	0.73	1.19	0.42	0.67
1	2½	4½	1	9.5	17.1	60	1.44	0.51	0.91	1.37	0.48	0.87	1.31	0.46	0.83	1.20	0.42	0.76	1.11	0.39	0.70
1	2½	5	1	9.5	19.0	54	1.37	0.48	0.96	1.30	0.46	0.92	1.24	0.44	0.87	1.13	0.40	0.80	1.04	0.37	0.73
1	2½	5½	1	9.5	20.9	49	1.30	0.46	1.01	1.23	0.43	0.95	1.17	0.41	0.91	1.07	0.38	0.83	0.98	0.34	0.76
1	2½	6	1	9.5	22.8	46	1.24	0.44	1.05	1.17	0.41	0.99	1.11	0.39	0.94	1.01	0.36	0.85	0.92	0.32	0.78
1	2½	6½	1	9.5	24.7	42	1.18	0.42	1.08	1.12	0.39	1.02	1.06	0.37	0.97	0.96	0.34	0.88	0.88	0.31	0.80
1	2½	7	1	9.5	26.6	40	1.13	0.40	1.11	1.07	0.38	1.05	1.01	0.36	0.99	0.91	0.32	0.90	0.83	0.29	0.82
1	3	4	1	11.4	15.2	76	1.42	0.60	0.80	1.36	0.57	0.77	1.30	0.55	0.73	1.21	0.51	0.68	1.12	0.47	0.63
1	3	4½	1	11.4	17.1	68	1.34	0.57	0.85	1.28	0.54	0.81	1.23	0.52	0.78	1.13	0.48	0.72	1.05	0.44	0.66
1	3	5	1	11.4	19.0	61	1.28	0.54	0.90	1.22	0.52	0.86	1.17	0.49	0.82	1.07	0.45	0.75	0.99	0.42	0.70
1	3	5½	1	11.4	20.9	56	1.22	0.52	0.94	1.16	0.49	0.90	1.11	0.47	0.86	1.01	0.43	0.78	0.93	0.39	0.72
1	3	6	1	11.4	22.8	52	1.16	0.49	0.98	1.11	0.47	0.94	1.05	0.44	0.89	0.96	0.41	0.81	0.88	0.37	0.74
1	3	6½	1	11.4	24.7	48	1.12	0.47	1.02	1.06	0.45	0.97	1.01	0.43	0.92	0.92	0.39	0.84	0.84	0.35	0.77
1	3	7	1	11.4	26.6	45	1.07	0.45	1.05	1.01	0.43	0.99	0.96	0.40	0.95	0.87	0.37	0.86	0.80	0.34	0.79
1	3	7½	1	11.4	28.5	42	1.03	0.44	1.09	0.97	0.41	1.02	0.92	0.39	0.97	0.83	0.35	0.88	0.76	0.32	0.80
1	3	8	1	11.4	30.4	40	0.99	0.42	1.11	0.93	0.39	1.05	0.88	0.37	0.99	0.80	0.34	0.90	0.73	0.31	0.82
1	4	5	1	15.2	19.0	76	1.13	0.64	0.80	1.08	0.61	0.76	1.04	0.59	0.73	0.96	0.54	0.68	0.90	0.51	0.63
1	4	6	1	15.2	22.8	64	1.04	0.59	0.88	0.99	0.56	0.84	0.95	0.54	0.80	0.87	0.49	0.73	0.81	0.46	0.68
1	4	7	1	15.2	26.6	55	0.96	0.54	0.95	0.92	0.52	0.91	0.88	0.50	0.87	0.80	0.45	0.79	0.74	0.42	0.73
1	4	8	1	15.2	30.4	49	0.90	0.51	1.01	0.85	0.48	0.96	0.81	0.46	0.91	0.74	0.42	0.83	0.68	0.38	0.77
1	4	9	1	15.2	34.2	44	0.84	0.47	1.06	0.80	0.45	1.01	0.76	0.43	0.96	0.68	0.38	0.86	0.63	0.35	0.80
1	4	10	1	15.2	38.0	40	0.79	0.44	1.11	0.75	0.42	1.06	0.71	0.40	1.00	0.64	0.36	0.90	0.58	0.33	0.82
1	5	10	1	19.0	38.0	47	0.73	0.52	1.03	0.69	0.49	0.97	0.66	0.46	0.93	0.60	0.42	0.84	0.55	0.39	0.77
1	6	12	1	22.8	45.5	46	0.62	0.52	1.04	0.58	0.49	0.98	0.56	0.47	0.94	0.50	0.42	0.84	0.46	0.39	0.78

NOTE.—Variations in the fineness of the sand and the compacting of the concrete may affect the quantities 10% in either direction.

*Use 50% columns for broken stone screened to uniform size.

†Use 45% columns for average conditions and for broken stone with **dust screened out.**

‡Use 40% columns for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

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QUANTITIES OF MATERIALS FOR ONE CUBIC YARD OF RAMMED CONCRETE. BASED ON A BARREL OF 4 CUBIC FEET.

(See important foot-notes, also p 225.)

PROPORTIONS BY PARTS			PROPORTIONS BY VOLUMES			Volume of mortar in terms of percentage of volume of stone	PERCENTAGES OF VOIDS IN BROKEN STONE OR GRAVEL															
							50%*			45%†			40%‡			30%§			20%§		
Cement	Sand	Stone	Packed Cement	Loose Sand	Loose Stone		Cement	Sand	Stone	Cement	Sand	Stone	Cement	Sand	Stone	Cement	Sand	Stone	Cement	Sand	Stone
			bbl.	cu. ft.	cu. ft.	%	bbl.	cu. yd.	cu. yd.	bbl	cd. yu	cu. yd.	bbl.	cu. yd.	cu. yd.	bbl	cu. yd.	u. d.	bbl.	cu yd	cu. yd.
1		1	1		4	89	4.99		0.74	4.80		0.71	4.62		0.69	4.23		0.63	3.91		0.53
1		2	1		8	49	3.57		1.06	3.37		1.00	3.20		0.95	2.84		0.84	2.56		0.76
1		3	1		12	35				2.60		1.16	2.45		1.09	2.13		0.95	1.90		0.84
1		4	1		16	28										1.71		1.01	1.51		0.89
1		5	1		20	24										1.43		1.06	1.26		0.93
1		6	1		24	22										1.22		1.08	1.07		0.95
1		7	1		28	20													0.94		0.98
1		8	1		32	18													0.83		0.98
1		9	1		36	17													0.75		1.00
1		10	1		40	16													0.68		1.01
1		11	1		44	15													0.62		1.01
1		12	1		48	15													0.57		1.01
1	1	1½	1	4	6	96	3.08	0.46	0.68	2.97	0.44	0.66	2.87	0.42	0.64	2.69	0.40	0.60	2.53	0.38	0.56
1	1	2	1	4	8	73	2.74	0.41	0.81	2.63	0.39	0.78	2.52	0.37	0.75	2.33	0.34	0.69	2.17	0.32	0.64
1	1	2½	1	4	10	59	2.47	0.37	0.91	2.35	0.35	0.87	2.25	0.33	0.83	2.06	0.31	0.76	1.90	0.28	0.71
1	1	3	1	4	12	50	2.25	0.33	1.00	2.13	0.32	0.95	2.03	0.30	0.90	1.85	0.27	0.82	1.70	0.25	0.76
1	1½	2	1	6	8	92	2.39	0.53	0.71	2.30	0.51	0.68	2.22	0.49	0.66	2.07	0.46	0.61	1.94	0.43	0.58
1	1½	2½	1	6	10	74	2.18	0.48	0.81	2.09	0.46	0.77	2.01	0.45	0.74	1.86	0.41	0.69	1.73	0.38	0.64
1	1½	3	1	6	12	62	2.01	0.45	0.89	1.91	0.42	0.85	1.83	0.41	0.81	1.68	0.37	0.75	1.56	0.35	0.69
1	1½	3½	1	6	14	54	1.86	0.41	0.96	1.77	0.39	0.92	1.68	0.37	0.87	1.54	0.34	0.80	1.42	0.32	0.74
1	1½	4	1	6	16	48	1.73	0.38	1.03	1.64	0.36	0.97	1.56	0.35	0.92	1.42	0.32	0.84	1.30	0.29	0.77
1	1½	4½	1	6	18	43	1.62	0.36	1.08	1.53	0.34	1.02	1.45	0.32	0.97	1.31	0.29	0.87	1.20	0.27	0.80
1	1½	5	1	6	20	39	1.52	0.34	1.13	1.43	0.32	1.06	1.35	0.30	1.00	1.22	0.27	0.90	1.11	0.25	0.82
1	2	3	1	8	12	74	1.81	0.54	0.80	1.74	0.52	0.77	1.67	0.50	0.74	1.54	0.46	0.68	1.44	0.43	0.64
1	2	3½	1	8	14	64	1.69	0.50	0.88	1.61	0.48	0.83	1.54	0.46	0.80	1.42	0.42	0.74	1.31	0.39	0.68
1	2	4	1	8	16	56	1.58	0.47	0.94	1.51	0.45	0.89	1.44	0.43	0.85	1.32	0.39	0.78	1.21	0.36	0.72
1	2	4½	1	8	18	51	1.49	0.44	0.99	1.41	0.42	0.94	1.34	0.40	0.89	1.23	0.36	0.82	1.13	0.34	0.75
1	2	5	1	8	20	46	1.40	0.42	1.04	1.33	0.39	0.98	1.26	0.37	0.93	1.15	0.34	0.85	1.05	0.31	0.78
1	2	5½	1	8	22	42	1.33	0.39	1.08	1.26	0.37	1.03	1.19	0.35	0.97	1.08	0.32	0.88	0.98	0.29	0.80
1	2	6	1	8	24	39	1.26	0.37	1.12	1.19	0.35	1.06	1.13	0.34	1.00	1.02	0.30	0.91	0.93	0.28	0.83
1	2½	3	1	10	12	86	1.65	0.61	0.73	1.59	0.59	0.71	1.53	0.57	0.68	1.42	0.52	0.63	1.33	0.49	0.59
1	2½	3½	1	10	14	75	1.55	0.57	0.80	1.48	0.55	0.77	1.42	0.52	0.74	1.32	0.49	0.68	1.23	0.46	0.64
1	2½	4	1	10	16	66	1.46	0.54	0.87	1.39	0.51	0.82	1.33	0.49	0.79	1.23	0.46	0.73	1.14	0.42	0.68
1	2½	4½	1	10	18	59	1.38	0.51	0.92	1.31	0.48	0.87	1.25	0.46	0.83	1.15	0.43	0.77	1.06	0.39	0.71
1	2½	5	1	10	20	54	1.31	0.48	0.97	1.24	0.46	0.92	1.18	0.44	0.87	1.08	0.40	0.80	0.99	0.37	0.73
1	2½	5½	1	10	22	49	1.24	0.46	1.01	1.18	0.44	0.96	1.12	0.41	0.91	1.02	0.38	0.83	0.93	0.34	0.76
1	2½	6	1	10	24	45	1.18	0.44	1.05	1.12	0.41	1.00	1.06	0.39	0.94	0.96	0.36	0.85	0.88	0.33	0.78
1	2½	6½	1	10	26	42	1.13	0.42	1.09	1.07	0.40	1.03	1.01	0.37	0.97	0.92	0.34	0.89	0.84	0.31	0.81
1	2½	7	1	10	28	39	1.08	0.40	1.12	1.02	0.38	1.06	0.96	0.36	1.00	0.87	0.32	0.90	0.79	0.29	0.82
1	3	4	1	12	16	75	1.35	0.60	0.80	1.30	0.58	0.77	1.25	0.56	0.74	1.15	0.51	0.68	1.08	0.48	0.64
1	3	4½	1	12	18	67	1.28	0.57	0.85	1.23	0.55	0.82	1.18	0.52	0.79	1.08	0.48	0.72	1.01	0.45	0.67
1	3	5	1	12	20	60	1.22	0.54	0.90	1.16	0.52	0.86	1.11	0.49	0.82	1.02	0.45	0.76	0.94	0.42	0.70
1	3	5½	1	12	22	55	1.16	0.52	0.95	1.11	0.49	0.90	1.06	0.47	0.86	0.97	0.43	0.79	0.89	0.40	0.72
1	3	6	1	12	24	50	1.11	0.49	0.99	1.06	0.47	0.94	1.01	0.45	0.90	0.92	0.41	0.82	0.84	0.37	0.75
1	3	6½	1	12	26	48	1.06	0.47	1.02	1.01	0.45	0.97	0.96	0.43	0.92	0.87	0.39	0.84	0.80	0.36	0.77
1	3	7	1	12	28	44	1.02	0.45	1.06	0.97	0.43	1.01	0.92	0.41	0.95	0.83	0.37	0.86	0.76	0.34	0.79
1	3	7½	1	12	30	42	0.98	0.44	1.09	0.93	0.41	1.03	0.88	0.39	0.98	0.79	0.35	0.88	0.73	0.32	0.81
1	3	8	1	12	32	39	0.94	0.42	1.11	0.89	0.40	1.05	0.84	0.37	1.00	0.76	0.34	0.90	0.69	0.31	0.82
1	4	5	1	16	20	75	1.08	0.64	0.80	1.03	0.61	0.76	0.99	0.59	0.73	0.92	0.55	0.68	0.86	0.51	0.64
1	4	6	1	16	24	63	0.99	0.59	0.88	0.95	0.56	0.84	0.91	0.54	0.81	0.83	0.49	0.74	0.77	0.46	0.68
1	4	7	1	16	28	55	0.92	0.54	0.95	0.88	0.52	0.91	0.83	0.49	0.86	0.76	0.45	0.79	0.70	0.42	0.73
1	4	8	1	16	32	48	0.86	0.51	1.02	0.81	0.48	0.96	0.77	0.46	0.91	0.70	0.42	0.83	0.64	0.38	0.76
1	4	9	1	16	36	43	0.80	0.47	1.07	0.76	0.45	1.01	0.72	0.43	0.96	0.65	0.39	0.87	0.60	0.36	0.80
1	4	10	1	16	40	40	0.75	0.44	1.11	0.71	0.42	1.05	0.67	0.40	0.99	0.61	0.36	0.90	0.55	0.33	0.81
1	5	10	1	20	40	47	0.70	0.52	1.04	0.66	0.49	0.98	0.63	0.47	0.93	0.57	0.42	0.84	0.52	0.38	0.77
1	6	12	1	24	48	46	0.59	0.52	1.05	0.56	0.50	1.00	0.53	0.47	0.94	0.48	0.43	0.85	0.44	0.39	0.78

NOTE.—Variations in the fineness of the sand and the compacting of the concrete may affect the quantities by the 10% in either direction.

*Use 50% columns for broken stone screened to uniform size.

†Use 45% columns for average conditions and for broken stone with dust screened out.

‡Use 40% columns for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

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VOLUME OF CONCRETE BASED ON A BARREL OF 3.5 CUBIC FEET.

(See important foot-notes, also p. 225.)

PROPORTIONS BY PARTS			PROPORTIONS BY VOLUME			Volume of mortar in terms of percentage of volume of stone	AVERAGE VOLUME OF RAMMED CONCRETE MADE FROM ONE BARREL CEMENT — Percentages of Voids in Broken Stone or Gravel				
Cement	Sand	Stone	Cement	Sand	Stone		50%*	45%†	40%‡	30%§	20%§
			bbl.	cu. ft.	cu. ft.	%	cu. ft.	cu. ft.	cu. ft	cu. ft.	cu. ft.
1		1	1		3.5	101	5.1	5.3	5.5	6.0	[illegible].4
1		2	1		7.0	54	7.0	7.4	7.8	8.7	9.6
1		3	1		10.5	39		9.5	10.0	11.5	12.8
1		4	1		14.0	31				14.2	16.0
1		5	1		17.5	27				17.0	19.2
1		6	1		21.0	24				19.7	22.4
1		7	1		24.5	21					25.6
1		8	1		28.0	20					28.8
1		9	1		31.5	18					32.0
1		10	1		35.0	17					35.2
1		11	1		38.5	16					38.4
1		12	1		42.0	16					41.6
1	1	1½	1	3.5	5.2	104	8.0	8.3	8.6	9.1	9.7
1	1	2	1	3.5	7.0	78	8.9	9.3	9.7	10.5	11.2
1	1	2½	1	3.5	8.7	64	9.9	10.4	10.8	11.8	12.7
1	1	3	1	3.5	10.5	54	10.8	11.4	12.0	13.1	14.2
1	1½	2	1	5.2	7.0	95	10.2	10.6	11.0	11.7	12.5
1	1½	2½	1	5.2	8.7	78	11.2	11.6	12.1	13.0	14.0
1	1½	3	1	5.2	10.5	65	12.1	12.7	13.2	14.4	15.5
1	1½	3½	1	5.2	12.2	56	13.0	13.7	14.4	15.7	17.0
1	1½	4	1	5.2	14.0	50	14.0	14.8	15.5	17.0	18.5
1	1½	4½	1	5.2	15.7	45	14.9	15.8	16.6	18.3	20.0
1	1½	5	1	5.2	17.5	41	15.9	16.8	17.8	20.0	21.6
1	2	3	1	7.0	10.5	77	13.4	13.9	14.5	15.6	16.8
1	2	3½	1	7.0	12.2	67	14 3	15.0	15.6	17.0	18.3
1	2	4	1	7.0	14.0	59	15.3	16.0	16.8	18.3	19.8
1	2	4½	1	7.0	15.7	53	16.2	17.0	17.9	19.6	21.3
1	2	5	1	7.0	17.5	48	17.1	18.1	19.0	20.9	22.8
1	2	5½	1	7.0	19.2	44	18.1	19.1	20.2	22.2	24.3
1	2	6	1	7.0	21.0	41	19.0	20.2	21.3	23.6	25.8
1	2½	3	1	8.7	10.5	90	14.6	15.2	15.8	16.9	18.0
1	2½	3½	1	8.7	12.2	78	15.6	16.2	16.9	18.2	19.6
1	2½	4	1	8.7	14.0	68	16.5	17.3	18.0	19.6	21.1
1	2½	4½	1	8.7	15.7	61	17.5	18.3	19.2	20.9	22.6
1	2½	5	1	8.7	17.5	55	18.4	19.4	20.3	22.2	24.1
1	2½	5½	1	8.7	19.2	51	19.4	20.4	21.4	23.5	25.6
1	2½	6	1	8.7	21.0	47	20.3	21.4	22.6	24.8	27.1
1	2½	6½	1	8.7	22.7	44	21.2	22.5	23.7	26.2	28.6
1	2½	7	1	8.7	24.5	41	22.2	23.5	24.8	27.5	30.1
1	3	4	1	10.5	14.0	77	17.8	18.5	19.3	20.8	22.3
1	3	4½	1	10.5	15.7	69	18.7	19.6	20.4	22.1	23.8
1	3	5	1	10.5	17.5	62	19.7	20.6	21.6	23.4	25.3
1	3	5½	1	10.5	19.2	57	20.6	21.7	22.7	24.8	26.8
1	3	6	1	10.5	21.0	53	21.6	22.7	23.8	26.1	28.4
1	3	6½	1	10.5	22.7	49	22.5	23.7	25.0	27.4	29.9
1	3	7	1	10.5	24.5	46	23.5	24.8	26.1	28.7	31.4
1	3	7½	1	10.5	26.2	43	24.4	25.8	27.2	30.1	32.9
1	3	8	1	10.5	28.0	40	25.3	26.9	28.4	31.4	34.4
1	4	5	1	14.0	17.5	77	22.2	23.2	24.1	26.0	27.9
1	4	6	1	14.0	21.0	65	24.1	25.2	26.4	28.6	30.9
1	4	7	1	14.0	24.5	56	26.0	27.3	28.6	31.3	33.9
1	4	8	1	14.0	28.0	50	27.9	39.4	30.9	33.9	36.9
1	4	9	1	14.0	31.5	45	29.8	31.5	33.2	36.6	40.0
1	4	10	1	14.0	35.0	41	31.7	33.6	35.4	39.2	43.0
1	5	10	1	17.5	35.0	48	34.2	36.1	38.0	41.8	45.5
1	6	12	1	21.0	42.0	46	40.5	42.8	45.0	49.6	54.1

NOTE.—Variations in the fineness of the sand and the compacting of the concrete may affect the volumes by 10% in either direction.

*Use 50% column for broken stone screened to uniform size.

†Use 45% column for average conditions and for broken stone with dust screened out.

‡Use 40% colums for gravel or mixed stone and gravel.

§Use these colums for scientifically graded mixtures.

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VOLUME OF CONCRETE BASED ON A BARREL OF 3.8 CUBIC FEET.

(See important foot-notes, also p. 225.)

PROPORTIONS BY PARTS			PROPORTIONS BY VOLUME			Volume of mortar in terms of percentage of volume of stone	AVERAGE VOLUME OF RAMMED CONCRETE MADE FROM ONE BARREL CEMENT				
							Percentages of Voids in Broken Stone or Gravel				
Cement	Sand	Stone	Cement	Sand	Stone		50%*	45%†	40%‡	30%§	20%§
			bbl	cu. ft.	cu. ft.	%	cu. ft.	cu. ft.	cu. ft.	cu. ft.	cu. ft.
1		1	1		3.8	94	5.3	5.5	5.7	6.2	6.7
1		2	1		7.6	51	7.4	7.8	8.2	9.2	10.2
1		3	1		11.4	36		10.0	10.6	12.2	13.6
1		4	1		15.2	29				15.2	17.1
1		5	1		19.0	25				18.2	20.6
1		6	1		22.8	22				21.1	24.0
1		7	1		26.6	20					27.5
1		8	1		30.4	19					31.0
1		9	1		34.2	18					34.4
1		10	1		38.0	17					37.9
1		11	1		41.8	16					41.4
1		12	1		45.5	15					44.8
1	1	1½	1	3.8	5.7	99	8.5	8.8	9.1	9.7	10.3
1	1	2	1	3.8	7.6	75	9.5	9.9	10.3	11.1	11.9
1	1	2½	1	3.8	9.5	61	10.5	10.0	11.5	12.6	13.6
1	1	3	1	3.8	11.4	51	11.5	12.2	12.8	14.0	15.2
1	1½	2	1	5.7	7.6	93	10.8	11.3	11.7	12.5	13.3
1	1½	2½	1	5.7	9.5	76	11.9	12.4	12.9	13.9	15.0
1	1½	3	1	5.7	11.4	64	12.9	13.5	14.1	15.4	16.6
1	1½	3½	1	5.7	13.3	55	13.9	14.6	15.4	16.8	18.2
1	1½	4	1	5.7	15.2	49	15.0	15.8	16.6	18.2	19.9
1	1½	4½	1	5.7	17.1	44	16.0	16.9	17.8	19.7	21.5
1	1½	5	1	5.7	19.0	40	17.0	18.0	19.1	21.1	23.2
1	2	3	1	7.6	11.4	75	14.3	14.9	15.5	16.7	18.0
1	2	3½	1	7.6	13.3	65	15.3	16.0	16.8	18.2	19.6
1	2	4	1	7.6	15.2	57	16.3	17.2	18.0	19.6	21.3
1	2	4½	1	7.6	17.1	51	17.4	18.3	19.2	21.0	22.9
1	2	5	1	7.6	19.0	47	18 4	19.4	20.4	22.5	24.5
1	2	5½	1	7.6	20.9	43	19.4	20.5	21.7	23.9	26.2
1	2	6	1	7.6	22.8	40	20.4	21.7	22.9	25.4	27.8
1	2½	3	1	9.5	11.4	87	15.7	16.3	16.9	18.1	19.3
1	2½	3½	1	9.5	13.3	75	16.7	17.4	18.1	19.6	21.0
1	2½	4	1	9.5	15.2	66	17.7	18.5	19.3	21.0	22.6
1	2½	4½	1	9.5	17.1	60	18.7	19.6	20.6	22.4	24.3
1	2½	5	1	9.5	19.0	54	19.8	20.8	21.8	23.9	25.9
1	2½	5½	1	9.5	20.9	49	20.8	21.9	23.0	25.3	27.6
1	2½	6	1	9.5	22.8	46	21.8	23.0	24.3	26.7	29.2
1	2½	6½	1	9.5	24.7	42	22.8	24.2	25.5	28.2	30.8
1	2½	7	1	9.5	26.6	40	23.9	25.3	26.7	29.6	32.5
1	3	4	1	11.4	15.2	76	19.1	19.9	20.7	22.4	24.0
1	3	4½	1	11.4	17.1	68	20.1	21.0	21.9	23.8	25.6
1	3	5	1	11.4	19.0	61	21.1	22.1	23.2	25.2	27.2
1	3	5½	1	11.4	20.9	56	22.1	23.3	24.4	26.7	28.9
1	3	6	1	11.4	22.8	52	23.2	24.4	25.6	28.1	30.6
1	3	6½	1	11.4	24.7	48	24.2	25.5	26.9	29.5	32.2
1	3	7	1	11.4	26.6	45	25.2	26.7	28.1	31.0	33.8
1	3	7½	1	11.4	28.5	42	26.2	27.8	29.3	32.4	35.5
1	3	8	1	11.4	30.4	40	27.3	28.9	30.6	33.8	37.1
1	4	5	1	15.2	19.0	76	23.9	24.9	25.9	28.0	30.0
1	4	6	1	15.2	22.8	64	25.9	27.2	28.4	30.8	33.3
1	4	7	1	15.2	26.6	55	28.0	29.4	30.8	33.7	36.6
1	4	8	1	15.2	30.4	49	30.0	31.7	33.3	36.6	39.9
1	4	9	1	15.2	34.2	44	32.1	33.9	35.8	39.4	43.1
1	4	10	1	15.2	38.0	40	34.1	36.2	38.2	42.3	46.4
1	5	10	1	19.0	38.0	47	36.9	38.9	41.0	45.1	49.2
1	6	12	1	22.8	45.5	46	43.7	46.2	48.6	53.6	58.5

NOTE.—Variations in the fineness of the sand and the compacting of the concrete may affec. the volumes by 10% in either direction.

*Use 50% column for broken stone screened to iniform size.

†**Use 45% column for average condRions** and for broken stone with dust screened out.

‡Use 40% column for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

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VOLUME OF CONCRETE BASED ON A BARREL OF 4 CUBIC FEET.

(See important foot-notes, also p. 225)

Proportions by Parts: Cement	Sand	Stone	Proportions by Volume: Cement	Sand	Stone	Volume of mortar in terms of percentage of volume of stone	Average Volume of Rammed Concrete made from One Barrel Cement — Percentages of Voids in Broken Stone or Gravel: 50%*	45%†	40%‡	30%§	20%§
			bbl.	cu. ft.	cu. ft.	%	cu. ft.	cu. ft.	cu ft.	cu. ft.	cu. ft.
1		1	1		4	89	5.4	5.6	5.8	6.4	6.9
1		2	1		8	49	7.6	8.0	8.4	9.5	10.5
1		3	1		12	35		10.4	11.0	12.7	14.2
1		4	1		16	28				15.8	17.8
1		5	1		20	24				18.9	21.5
1		6	1		24	22				22.1	25.1
1		7	1		28	20					28.8
1		8	1		32	18					32.4
1		9	1		36	17					36.1
1		10	1		40	16					39.7
1		11	1		44	15					43.4
1		12	1		48	15					47.0
1	1	1½	1	4	6	96	8.8	9.1	9.4	10.0	10.7
1	1	2	1	4	8	73	9.8	10.3	10.7	11.6	12.4
1	1	2½	1	4	10	59	10.9	11.5	12.0	13.1	14.2
1	1	3	1	4	12	50	12.0	12.7	13.3	14.6	15.9
1	1½	2	1	6	8	92	11.3	11.7	12.2	13.0	13.9
1	1½	2½	1	6	10	74	12.4	12.9	13.5	14.5	15.6
1	1½	3	1	6	12	62	13.5	14.1	14.8	16.0	17.3
1	1½	3½	1	6	14	54	14.5	15.3	16.0	17.6	19.1
1	1½	4	1	6	16	48	15.6	16.5	17.3	19.1	20.8
1	1½	4½	1	6	18	43	16.7	17.7	18.6	20.6	22.5
1	1½	5	1	6	20	39	17.8	18.9	19.9	22.1	24.3
1	2	3	1	8	12	74	14.9	15.6	16.2	17.5	18.8
1	2	3½	1	8	14	64	16.0	16.7	17.5	19.0	20.5
1	2	4	1	8	16	56	17.1	17.9	18.8	20.5	22.3
1	2	4½	1	8	18	51	18.1	19.1	20.1	22.0	23.9
1	2	5	1	8	20	46	19.2	20.3	21.4	23.5	25.7
1	2	5½	1	8	22	42	20.3	21.5	22.7	25.1	27.4
1	2	6	1	8	24	39	21.4	22.7	24.0	26.6	29.2
1	2½	3	1	10	12	86	16.3	17.0	17.6	18.9	20.2
1	2½	3½	1	10	14	75	17.4	18.2	18.9	20.5	22.0
1	2½	4	1	10	16	66	18.5	19.4	20.2	21.9	23.7
1	2½	4½	1	10	18	50	19.6	20.6	21.5	23.5	25.4
1	2½	5	1	10	20	54	20.7	21.8	22.8	25.0	27.2
1	2½	5½	1	10	22	49	21.8	22.9	24.1	26.5	28.9
1	2½	6	1	10	24	45	22.8	24.1	25.4	28.0	30.6
1	2½	6½	1	10	26	42	23.9	25.3	26.7	29.5	32.3
1	2½	7	1	10	28	39	25.0	26.5	28.0	31.0	34.0
1	3	4	1	12	16	75	20.0	20.8	21.7	23.4	25.1
1	3	4½	1	12	18	67	21.0	22.0	23.0	24.9	26.8
1	3	5	1	12	20	60	22.1	23.2	24.3	26.4	28.6
1	3	5½	1	12	22	55	23.2	24.4	25.6	28.0	30.3
1	3	6	1	12	24	50	24.3	25.6	26.9	29.5	32.1
1	3	6½	1	12	26	48	25.4	26.8	28.2	31.0	33.8
1	3	7	1	12	28	44	26.4	27.9	29.4	32.5	35.5
1	3	7½	1	12	30	42	27.5	29.1	30.8	34.0	37.2
1	3	8	1	12	32	39	28.6	30.3	32.0	35.5	39.0
1	4	5	1	16	20	75	25.0	26.1	27.2	29.3	31.5
1	4	6	1	16	24	63	27.2	28.5	29.8	32.4	35.0
1	4	7	1	16	28	55	29.3	30.8	32.4	35.4	38.4
1	4	8	1	16	32	48	31.5	33.2	34.9	38.4	41.9
1	4	9	1	16	36	43	33.6	35.6	37.5	41.4	45.3
1	4	10	1	16	40	40	35.8	38.0	40.1	44.4	48.8
1	5	10	1	20	40	47	38.7	40.9	43.0	47.3	51.7
1	6	12	1	24	48	46	45.9	48.5	51.1	56.3	61.4

NOTE.—Variations in the fineness of the sand and the compacting of the concrete may affect the volumes by 10% in either direction.

*Use 50% column for broken stone screened to uniform size.

†Use 45% column for average conditions and for broken stone with dust screened out.

‡Use 40% column for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

THEORY OF REINFORCED CONCRETE.

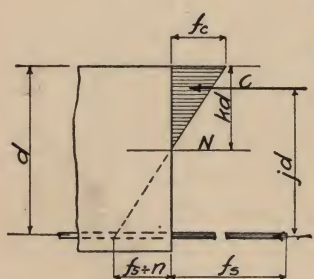
It is not our intention to give here a complete mathematical analysis of the stresses which occur in reinforced concrete structures. For such an analysis we would refer to any of the standard text books on the theory of reinforced concrete, where the subject is treated to a much larger scale than would be possible in the scope of this book.

The following formulas are now recommended by practically all the best authorities on reinforced concrete. They are based on the "straight line," or linear distribution of stress:

Notation.

F_s = unit fibre stress in steel in pounds per square inch.
F_c = unit fibre stress in concrete, extreme fibre in compression.
E_s = modulus of elasticity of the steel.
E_c = modulus of elasticity of the concrete.

$n = \text{ratio } \dfrac{E_s}{E_c}$



M_s = resisting moment in inch pounds due to steel.
M_c = resisting moment in inch pounds due to concrete.
b = breadth in inches of rectangular beam.
d = depth in inches of center of steel from compression face of concrete.
k = ratio of depth of neutral axis below the top to the effective depth d.
j = ratio of arm of resisting couple to depth d.

A = area in square inches of cross section of steel.

p = steel ratio (not percent) $= \frac{A}{bd}$

The position of the neutral axis is shown by the formula

$$K = \sqrt{2pn + (pn)^2} - pn$$

The lever arm of the resisting couple is shown by

$$j = 1 - \tfrac{1}{3}k$$

Resisting Moment as governed by the steel,

$$M_s = f_s pjbd^2$$

Resisting Moment as governed by the concrete,

$$M_c = \tfrac{1}{2} f_c kjbd^2$$

For approximate results use the average values, $k = \frac{3}{8}$ and $j = \frac{7}{8}$, which gives

$$M_s = \tfrac{7}{8}\, dAf_s \ldots. (1)$$
$$M_c = \tfrac{1}{6}\, bd^2f_c \ldots. (2)$$

Both moments should be figured and the lesser one used.

Tee-beams.

The same formulae hold good for the tee-beams except that the breadth (b) of the beam is in this case the width of the flange and not the width of the stem. Where tee-beams are formed by casting the rectangular beams monolithic with the floor slab, the resisting moment will in ordinary construction depend on the resistance of the steel in tension, so that it will only be necessary to use the formula for (M_s) in order to determine the resisting moment of the section.

Bending Moments.

If slabs and girders be reinforced to take care of negative bending moments over supports, they will act as continuous beams, and the bending moment at the center of the span will be reduced. It is considered good practice to use the following values:

For beams and slabs continuous over both supports,

$$M = \frac{1}{12} wl^2$$

Continuous over one support only,

$$M = \frac{1}{10} wl^2$$

Freely supported,

$$M = \frac{1}{8} wl^2$$

When M = Bending Moment at center of span in foot pounds.

w = total uniform live and dead load in pounds per square foot.

l = length of span in feet. When these moments are substituted in equation (1) and (2) they are to be multiplied by 12 to reduce the moment to inch pounds.

Unless care be taken to insure proper position of steel over supports, we would recommend using $M = \frac{1}{10} wl^2$.

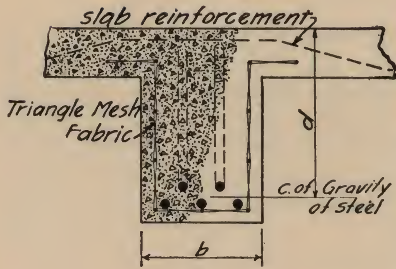
Shear in Rectangular Beams.

Let V = total shear at the sections in pounds.
b = width of section in inches.
d = depth of section to center of steel in inches.
v = unit shear in pounds per square inch.

then $v = \frac{V}{bd}$

With beams having no web reinforcement the working stresses should not exceed 35 to 40 pounds per square inch of cross section area of the concrete above the plane of the horizontal steel. If the beam has sufficient web reinforcement the working stresses may be taken at 100 to 125 pounds per square inch.

The web reinforcement may be supplied by using either vertical or inclined stirrups, or by bending up a portion of the main tension bars,



Beam Section.

or both combined. By using Triangle Mesh Wire Fabric in the form of a cage as shown by the accompanying cut the cross wires will give sufficient web reinforcement for all ordinary cases. By referring to the tables on pp. 110 and 111 it is seen that these cross wires may be either No. 14 or No. 12½ gauge spaced either 2″ or 4″ apart, thus giving a variation in amount of web reinforcement. This material, having a joint at each longitudinal, can be placed in forms at a very small expense as compared with loose stirrups. Another point in favor of Triangle Mesh fabric for web reinforcement is the truss action which this material gives, thus adding another factor of safety to the structure.

In cases of tee-beams it is necessary that the web reinforcement be carried well up into the slab to prevent shearing off on a plane between the flange and stem.

EXPLANATION OF TABLES FOR REINFORCED CONCRETE SLABS (PP. 74-86).

The following tables are based on the "straight line" formula, or linear distribution of stress as shown on page 66. The ratio of the modulus of elasticity of steel to concrete is taken as fifteen. Values of resisting moments of slabs are given per foot of width for various maximum values for steel and concrete. Below and to the left of the heavy zigzag line values of Resisting Moments are given as governed by the maximum allowable fibre stress in steel; the values above and to the right of this line are governed by maximum allowable fibre stress in concrete. The various values for maximum fibre stresses are thus given so that almost any specifications may be complied with.

The tables have been arranged in such a manner that a uniform reinforcement may be used and by increasing or decreasing the thickness of the slabs, spans of greater or less length than the average spans of the floor may be taken care of economically with the same reinforcement.

The second column gives the distance in inches from the center of the steel to the bottom of the slab. The third column gives the weight of the concrete slab per square foot of floor area, this weight being based on concrete weighing 144 pounds per cubic foot. Although in the following examples, we have used the formula $B.M. = \frac{Wl^2}{10}$, it is considered good practice to use $B.M. = \frac{Wl^2}{12}$ when the slab is continuous over both supports; however, care must be taken to have the reinforcement near the top of the slab over supports in order to resist the negative bending moments at these points:

Examples of the use of these tables:

Given:

- A live load of 75 pounds per square foot.
- Span of slab eight feet.
- Floor—cement finish of slab—no plaster below.
- Maximum allowable fibre stress in steel 18,000 pounds per square inch.
- Maximum allowable fibre stress in concrete 650 pounds per square inch.

Look in the Table No. 13 giving minimum thickness of slabs for various spans. From this table we find for a load of 100 pounds per square foot and span of eight feet, a minimum thickness of 3½ inches is recommended. It is better, however, to make this slab 4 inches thick.

The total dead load consists of the 4 inch slab, which will weigh

	48	pounds per square foot.
Live loads	75	pounds per square foot.
Total	123	pounds per square foot.

For slabs continuous over supports figure the bending moment as follows: (Or use Diagrams No. 1 or No. 2, pp. 88 and 89).

Bending Moment in foot pounds is equal to the total load per square foot multiplied by the span of the slab (in feet) squared and divided by ten. Or expressed as a formula (see article on Bending Moments page 67).

$$B.\,M. = \frac{wl^2}{10}$$

In which

B. M. = Bending moment in foot pounds.
w = Total load in pounds per square foot.
l = Length of span in feet.

For this particular example—

w = 123 (pounds per square foot).
l = 8 (feet span of slab).

Then

$$B.\,M. = \frac{123 \times 8^2}{10} = 787 \text{ foot pounds.}$$

Now it is necessary to find in Table No. 7 (corresponding to the given allowable stresses) the reinforcement for a four inch slab that will resist a bending moment of 787 foot pounds.

In this table we find a cross sectional area of steel of 0.18 square inches per foot width of slab will be needed to make a 4 inch slab capable of resisting a bending moment of this amount.

This area will be supplied with our Triangle Mesh Reinforcement Style number (33) as shown in the next to the last column in the tables p. 110; or the slab thickness can be changed back to $3\frac{1}{2}$ inches, in which case approximately 0.22 square inches will be required (exact amount may be determined by deducting 6 pounds from the total load and again determining the B. M.). Style No. 32 having a cross sectional area of 0.225 square inches per foot width will give the necessary steel area.

Slabs Reinforced in Two Directions.

It is very often desirable to reinforce the floor slab in two directions, and support the slab by means of beams (or walls) on four sides. If the panel is square then one-half of the total load is carried in each direction. In this case the bending moment due to the total load will be:

$$B.\,M. = \frac{1}{20} \times w \times l^2$$

Example:

Given: A live load of 75 pounds per square foot; size of panel 12′ x 12′; floor—cement finish of slab; maximum allowable fibre stress in steel 18,000 pounds per square inch; fibre stress in concrete 650 pounds per square inch; bending moment $= \frac{wl^2}{20}$.

We will assume a 4 inch slab. In this example w = live load of 75 pounds plus weight of a 4 inch slab or 48 pounds, giving a total load of 123 pounds per square foot; (l) = 12 feet, therefore

$$\text{B. M.} = \frac{123 \times (12)^2}{20} = 886 \text{ foot pounds.}$$

Now, referring to Table No. 7, in order to find the reinforcement for a four inch slab that will resist a bending moment of 886 foot pounds, we find a cross sectional area of steel of 0.205 square inches per foot width of slab will be needed to make a 4 inch slab capable of resisting a bending moment of 886 foot pounds. This area will be supplied with our Triangle Mesh Reinforcement, Style No. 41, (see p. 110) one layer placed in each direction.

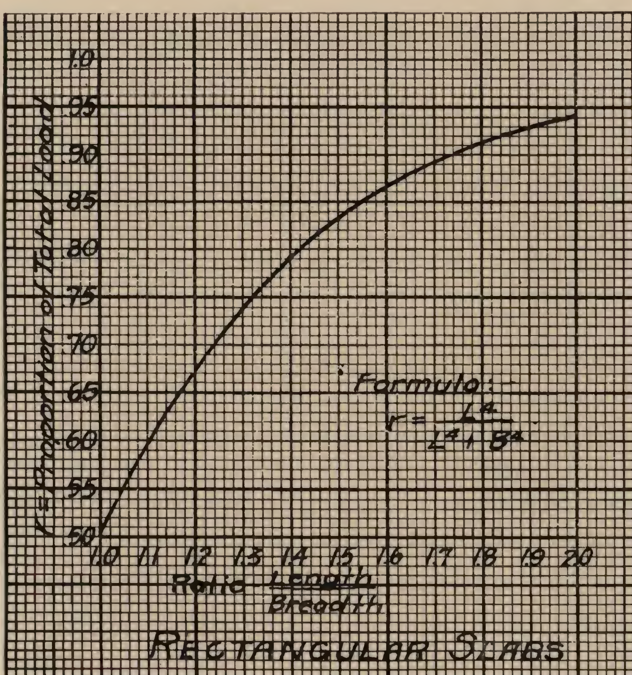
Panels Having Greater Length Than Breadth.

The distribution of load is first determined by the formula

$$r = \frac{l^4}{l^4 + b^4}$$

in which r equals proportion of load carried by the reinforcement placed the short way of the slab, l = length and b = breadth of slab.

The following diagram shows the value of r for various ratios of $\frac{\text{length}}{\text{breadth}}$



Rectangular Slabs.

Example:

Given: A live load of 75 pounds per square foot; size of panel 12′ x 10′; floor—cement finish of slab; maximum allowable fibre stress in steel 18,000 pounds per square inch; maximum allowable fibre stress in concrete 650 pounds per square inch.

In this example we will assume a 4 inch slab, then $w = 75$ pounds live load plus weight of a 4 inch slab or 48 pounds, giving a total load of 123 pounds per square foot.

Now, to determine what portions of this load are carried by the two systems of reinforcement, divide the length of panel by breadth, or

$$\frac{l}{b} = \frac{12}{10} = 1.2$$

Now, with this value for $\frac{l}{b}$ we will enter the diagram for Rectangular slabs (p. 71). We find the value of $\frac{l}{b} = 1.2$ and follow the vertical line to the point of its intersection with the curved line; now following the horizontal line to the left we find that $\frac{67}{100}$ of the total load is carried by the short span reinforcement; then necessarily the remaining part or $\frac{33}{100}$ of the total load is carried by the long span reinforcement. The two systems of reinforcement are now treated as separate problems.

The Bending Moment for the short span = B. M. = $\frac{1}{10}$ × load × span squared.

But load = $\frac{67}{100} \times 123$ pounds = 83 pounds per square foot; therefore

$$\text{B. M.} = \frac{83 \times 10^2}{10} = 830 \text{ foot pounds.}$$

Referring to Table No. 7 to find the reinforcement for a 4 inch slab that will resist a B. M. of 830 foot pounds. We find a cross sectional area of steel of 0.19 square inches per foot width of slab will be needed to make a 4 inch slab capable of resisting a bending moment of 830 foot pounds. This can be supplied by our Style No. 33. (See p. 110.)

The Bending Moment for the long span =

B. M. = $\frac{1}{10}$ × load × span squared. But load = $\frac{33}{100} \times 123 = 40$ pounds per square foot.

Therefore $$\text{B. M.} = \frac{40 \times 12^2}{10} = 576 \text{ foot pounds.}$$

Again referring to Table No. 7 we find a cross sectional area of steel of 0.125 square inches per foot width will be needed to make a 4 inch slab capable of resisting a bending moment of 576 foot pounds. This can be supplied by our style No. 25. (See p. 110.)

Note: When a sectional area of steel is required which is greater than can be supplied by one layer of our heaviest material, either use two layers of fabric, or one layer of fabric with a sufficient number of loose bars to make up the required area.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq. ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \text{Bending Moment for *square slab supported on four sides.}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line.

For examples showing use of tables see page 69

Table No. 1 **Maximum Stresses: Steel = 16,000 pounds, Concrete = 500 pounds** **Conc. 1:3:6**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH — CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																						
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	86	130	168	210	229	242	252	262	271	290													
3	¾	36	114	165	222	271	327	366	383	400	413	445	470												
3½	¾	42	137	203	268	332	395	458	520	558	578	623	660	692											
4	¾	48	160	237	329	404	478	552	625	697	759	821	874	919	958										
4½	¾	54	192	275	377	458	557	636	734	812	890	1038	1111	1168	1219	1264									
5	1	60		313	407	498	589	679	769	858	968	1158	1231	1301	1357	1411	1456								
5½	1	66		337	455	572	659	774	888	973	1086	1337	1504	1583	1654	1717	1780	1832							
6	1	72			489	634	742	849	991	1095	1201	1513	1784	1880	1975	2051	2120	2190	2248						
6½	1	78			547	678	811	941	1071	1199	1327	1664	1957	2204	2314	2402	2492	2565	2641	2711					
7	1	84				756	913	1017	1172	1326	1478	1831	2179	2524	2674	2770	2886	2979	3052	3132	3206	3276			
7½	1¼	90				764	934	1103	1216	1383	1548	1877	2257	2632	2846	2972	3086	3176	3273	3362	3434	3512			
8	1¼	96					1023	1156	1352	1483	1678	2062	2443	2820	3252	3385	3506	3617	3721	3817	3906	4005	4084	4229	4360
8½	1¼	102					1104	1257	1409	1636	1786	2232	2673	3037	3470	3804	3929	4067	4194	4311	4404	4508	4605	4770	4931
9	1½	108						1346	1508	1670	1831	2309	2703	3172	3560	4015	4179	4303	4444	4552	4674	4769	4877	5058	5221
9½	1½	114						1437	1623	1807	1992	2447	2897	3343	3874	4313	4622	4784	4925	5082	5203	5318	5427	5644	5824
10	1½	120							1728	1936	2144	2660	3068	3573	4075	4572	5066	5300	5465	5620	5765	5874	6028	6242	6456

Note :—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq. ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \text{Bending Moment for *square slab supported on four sides.}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line. For examples showing use of tables see page 69

Table No. 2 **Maximum Stresses: Steel = 16,000 pounds, Concrete = 600 pounds** **Conc. 1:2½:5**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH																						
			CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																						
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	86	130	168	210	248	289	302	315	325	348													
3	¾	36	114	165	222	271	327	375	423	478	496	534	564												
3½	¾	42	137	203	268	332	395	458	520	592	653	747	792	830											
4	¾	48	160	237	329	404	478	552	625	697	769	954	1048	1103	1150										
4½	¾	54	192	275	377	458	557	636	734	812	890	1100	1327	1402	1463	1517									
5	1	60		313	407	498	589	679	769	858	968	1187	1403	1561	1627	1693	1747								
5½	1	66		337	455	572	659	774	888	973	1086	1337	1612	1857	1984	2060	2136	2198							
6	1	72			489	634	742	849	991	1095	1201	1513	1787	2058	2359	2461	2544	2628	2698						
6½	1	78			547	678	811	941	1071	1199	1327	1664	1957	2286	2612	2882	2990	3078	3169	3253					
7	1	84				756	913	1017	1172	1326	1478	1831	2179	2524	2866	3157	3463	3574	3663	3759	3848	3931			
7½	1¼	90				764	934	1103	1216	1383	1548	1877	2257	2632	2951	3320	3686	3810	3927	4034	4120	4215			
8	1¼	96					1023	1156	1352	1483	1678	2062	2443	2820	3257	3627	3995	4341	4465	4581	4689	4806	4900	5075	5232
8½	1¼	102					1104	1257	1409	1636	1786	2232	2673	3037	3470	3900	4256	4679	5032	5174	5284	5409	5526	5724	5917
9	1½	108						1346	1508	1670	1831	2309	2703	3172	3560	4021	4479	4857	5308	5462	5609	5723	5852	6069	6266
9½	1½	114						1437	1623	1807	1992	2447	2897	3343	3874	4313	4749	5182	5612	6099	6243	6381	6511	6773	6989
10	1½	120							1728	1936	2144	2660	3068	3573	4075	4572	5066	5557	6044	6529	6917	7048	7234	7490	7747

Note:—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq. ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \text{Bending Moment for *square slab supported on four sides.}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line. For example showing use of tables see page 69

Table No. 3 **Maximum Stresses: Steel = 16,000 pounds, Concrete = 650 pounds** **Conc. { 1:2: 4 / 1:2½:5 carefully graded**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH — CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																						
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	86	130	168	210	248	289	325	341	353	377													
3	¾	36	114	165	222	271	327	375	423	478	525	578	611												
3½	¾	42	137	203	268	332	395	458	520	592	653	804	858	900											
4	¾	48	160	237	329	404	478	552	625	697	769	954	1136	1194	1246										
4½	¾	54	192	275	377	458	557	636	734	812	890	1100	1327	1519	1585	1644									
5	1	60		313	407	498	589	679	769	858	968	1187	1403	1637	1764	1835	1893								
5½	1	66		337	455	572	659	774	888	973	1086	1337	1612	1857	2099	2232	2314	2381							
6	1	72			489	634	742	849	991	1095	1201	1513	1787	2058	2359	2625	2756	2848	2922						
6½	1	78			547	678	811	941	1071	1199	1327	1664	1957	2286	2612	2895	3216	3334	3431	3525					
7	1	84				756	913	1017	1172	1326	1478	1831	2179	2524	2866	3157	3541	3872	3968	4072	4169	4259			
7½	1¼	90				764	934	1103	1216	1383	1548	1877	2257	2632	2951	3320	3686	3998	4255	4371	4464	4566			
8	1¼	96					1023	1156	1352	1483	1678	2062	2443	2820	3257	3627	3995	4360	4723	4963	5080	5207	5309	5498	5668
8½	1¼	102					1104	1257	1409	1636	1786	2232	2673	3037	3470	3900	4256	4679	5100	5519	5725	5860	5987	6201	6410
9	1½	108						1346	1508	1670	1831	2309	2703	3172	3560	4021	4479	4857	5308	5683	6077	6200	6340	6576	6788
9½	1½	114						1437	1623	1807	1992	2447	2897	3343	3874	4313	4749	5182	5612	6125	6550	6914	7055	7338	7571
10	1½	120							1728	1936	2144	2660	3068	3573	4075	4572	5066	5557	6044	6529	7011	7395	7836	8114	8393

Note:—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq. ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \text{Bending Moment for *square slab supported on four sides.}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line. For example showing use of tables see page 69

Table No. 4 **Maximum Stresses: Steel = 16,000 pounds, Concrete = 700 pounds** **Conc. 1:2:4**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH — CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																						
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	86	130	168	210	248	289	325	366	379	406													
3	¾	36	114	165	222	271	327	375	423	478	525	623	658												
3½	¾	42	137	203	268	332	395	458	520	592	653	804	924	969											
4	¾	48	160	237	329	404	478	552	625	697	769	954	1137	1286	1342										
4½	¾	54	192	275	377	458	557	636	734	812	890	1100	1327	1532	1707	1770									
5	1	60		313	407	498	589	679	769	858	968	1187	1403	1637	1849	1976	2038								
5½	1	66		337	455	572	659	774	888	973	1086	1337	1612	1857	2099	2340	2492	2565							
6	1	72			489	634	742	849	991	1095	1201	1513	1787	2058	2359	2625	2889	3066	3147						
6½	1	78			547	678	811	941	1071	1199	1327	1664	1957	2286	2612	2895	3216	3495	3698	3796					
7	1	84				756	913	1017	1172	1326	1478	1831	2179	2524	2866	3157	3541	3875	4160	4386	4490	4586			
7½	1¼	90				764	934	1103	1216	1383	1548	1877	2257	2632	2951	3320	3686	3998	4359	4704	4802	4913			
8	1¼	96					1023	1156	1352	1483	1678	2062	2443	2820	3257	3627	3995	4360	4723	5084	5444	5607	5717	5920	6104
8½	1¼	102					1104	1257	1409	1636	1786	2232	2673	3037	3470	3900	4256	4679	5100	5519	5866	6280	6447	6678	6903
9	1½	108						1346	1508	1670	1831	2309	2703	3172	3560	4021	4479	4857	5308	5683	6129	6500	6828	7081	7310
9½	1½	114						1437	1623	1807	1992	2447	2897	3343	3874	4313	4749	5182	5612	6125	6550	6973	7395	7902	8154
10	1½	120							1728	1936	2144	2660	3068	3573	4075	4572	5066	5557	6044	6529	7011	7395	7968	8739	9038

Note:—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq. ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \text{Bending Moment for *square slab supported on four sides.}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line.

For example showing use of tables see page 69

Table No. 5 **Maximum Stresses: Steel = 18,000 pounds, Concrete = 500 pounds** **Conc. 1:3:6**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH																					
			CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																					
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	97	146	189	215	229	242	252	262	271	290													
3	¾	36	128	185	250	305	348	366	383	400	413	445	470												
3½	¾	42	154	228	301	373	444	506	532	558	578	623	660	692											
4	¾	48	180	267	370	454	538	621	702	731	759	821	874	919	958										
4½	¾	54	216	309	424	515	627	716	826	914	958	1038	1111	1168	1219	1264									
5	1	60		352	456	560	663	764	865	965	1072	1158	1231	1301	1357	1411	1456								
5½	1	66		379	512	644	741	871	999	1095	1222	1402	1504	1583	1654	1717	1780	1832							
6	1	72			550	714	835	956	1115	1234	1352	1675	1784	1880	1975	2051	2120	2190	2248						
6½	1	78			616	765	913	1059	1206	1349	1493	1872	2079	2204	2314	2402	2492	2565	2641	2711					
7	1	84				851	1027	1144	1318	1491	1663	2059	2412	2551	2674	2770	2886	2979	3052	3132	3206	3276			
7½	1¼	90				859	1050	1240	1366	1554	1741	2110	2537	2727	2846	2972	3086	3176	3273	3362	3434	3512			
8	1¼	96					1152	1300	1522	1668	1887	2320	2749	3079	3252	3385	3506	3617	3721	3817	3906	4005	4084	4229	4360
8½	1¼	102					1243	1414	1586	1840	2009	2511	3006	3416	3640	3804	3929	4067	4194	4311	4404	4508	4605	4770	4931
9	1½	108						1514	1697	1879	2059	2598	3041	3569	3834	4015	4179	4303	4444	4552	4674	4769	4877	5058	5221
9½	1½	114						1618	1827	2034	2240	2753	3259	3761	4288	4468	4622	4784	4925	5082	5203	5318	5427	5644	5824
10	1½	120							1944	2180	2413	2993	3451	4020	4584	4925	5120	5300	5465	5620	5765	5874	6028	6242	6456

Note:—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq. ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \text{Bending Moment for *square slab supported on four sides}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line. For examples showing use of tables see page 69

Table No. 6 **Maximum Stresses: Steel = 18,000 pounds, Concrete = 600 pounds** **Conc. 1:2½:5**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH																						
			CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																						
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	97	146	189	237	275	290	302	315	325	348													
3	¾	36	128	185	250	305	368	422	459	480	496	534	564												
3½	¾	42	154	228	301	373	444	515	585	666	694	747	792	830											
4	¾	48	180	267	370	454	538	621	703	784	865	986	1048	1103	1150										
4½	¾	54	216	309	424	515	627	716	826	914	1001	1238	1333	1402	1463	1517									
5	1	60		352	456	560	663	764	865	965	1090	1336	1477	1561	1627	1693	1747								
5½	1	66		379	512	644	741	871	999	1095	1222	1504	1805	1900	1984	2060	2136	2198							
6	1	72			550	714	835	956	1115	1234	1352	1702	2010	2256	2370	2461	2544	2628	2698						
6½	1	78			616	765	913	1059	1206	1349	1493	1872	2200	2571	2775	2882	2990	3078	3169	3253					
7	1	84				851	1027	1144	1318	1491	1663	2059	2452	2840	3209	3324	3463	3574	3663	3759	3848	3931			
7½	1¼	90				859	1050	1240	1366	1554	1741	2110	2537	2959	3318	3566	3703	3810	3927	4034	4120	4215			
8	1¼	96					1152	1300	1522	1668	1887	2320	2749	3173	3664	4061	4207	4341	4465	4581	4689	4806	4900	5075	5232
8½	1¼	102					1243	1414	1586	1840	2009	2511	3006	3416	3906	4386	4715	4880	5032	5174	5284	5409	5526	5724	5917
9	1½	108						1514	1697	1879	2059	2598	3041	3569	4004	4524	5014	5164	5333	5462	5609	5723	5852	6069	6266
9½	1½	114						1618	1827	2034	2240	2753	3259	3761	4358	4852	5342	5741	5910	6099	6243	6381	6511	6773	6989
10	1½	120							1944	2180	2413	2993	3451	4020	4584	5144	5700	6252	6557	6745	6917	7048	7234	7490	7747

Note:—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq. ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \text{Bending Moment for *square slab supported on four sides.}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line.

For example showing use of tables see page 69

Table No. 7 **Maximum Stresses: Steel = 18,000 pounds, Concrete = 650 pounds** **Conc. { 1:2: 4 / 1:2½:5 carefully graded**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH — CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																						
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	97	146	189	237	279	314	328	341	353	377													
3	¾	36	128	185	250	305	368	422	476	520	537	578	611												
3½	¾	42	154	228	301	373	444	515	585	666	734	810	858	900											
4	¾	48	180	267	370	454	538	621	703	784	865	1068	1136	1194	1246										
4½	¾	54	216	309	424	515	627	716	826	914	1001	1238	1444	1519	1585	1644									
5	1	60		352	456	560	663	764	865	965	1090	1336	1578	1691	1764	1835	1893								
5½	1	66		379	512	644	741	871	999	1095	1222	1504	1814	2058	2150	2232	2314	2381							
6	1	72			550	714	835	956	1115	1234	1352	1702	2010	2315	2568	2666	2756	2848	2922						
6½	1	78			616	765	913	1059	1206	1349	1493	1872	2200	2571	2938	3123	3240	3334	3431	3525					
7	1	84				851	1027	1144	1318	1491	1663	2059	2452	2840	3225	3551	3752	3872	3968	4072	4169	4259			
7½	1¼	90				859	1050	1240	1366	1554	1741	2110	2537	2959	3318	3735	4012	4128	4255	4371	4464	4566			
8	1¼	96					1152	1300	1522	1668	1887	2320	2749	3173	3664	4081	4494	4703	4837	4963	5080	5207	5309	5498	5668
8½	1¼	102					1243	1414	1586	1840	2009	2511	3006	3416	3906	4386	4789	5264	5452	5605	5725	5860	5987	6201	6410
9	1½	108						1514	1697	1879	2059	2598	3041	3569	4004	4524	5038	5464	5777	5918	6077	6200	6340	6576	6788
9½	1½	114						1618	1827	2034	2240	2753	3259	3761	4358	4852	5342	5830	6313	6607	6764	6914	7055	7338	7571
10	1½	120							1944	2180	2413	2993	3451	4020	4584	5144	5700	6252	6810	7307	7494	7636	7836	8114	8393

Note:—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq. ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \text{Bending Moment for *square slab supported on four sides.}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line.

For example showing use of tables see page 69

Table No. 8 **Maximum Stresses: Steel = 18,000 pounds, Concrete = 700 pounds** **Conc. 1:2:4**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH — CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																						
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	97	146	189	237	279	325	353	367	379	406													
3	¾	36	128	185	250	305	368	422	476	537	579	623	658												
3½	¾	42	154	228	301	373	444	515	585	666	734	872	924	969											
4	¾	48	180	267	370	454	538	621	703	784	865	1074	1223	1286	1342										
4½	¾	54	216	309	424	515	627	716	826	914	1001	1238	1493	1636	1707	1770									
5	1	60		352	456	560	663	764	865	965	1090	1336	1578	1821	1899	1976	2038								
5½	1	66		379	512	644	741	871	999	1095	1222	1504	1814	2089	2316	2404	2492	2563							
6	1	72			550	714	835	956	1115	1234	1352	1702	2010	2315	2654	2871	2968	3066	3147						
6½	1	78			616	765	913	1059	1206	1349	1493	1872	2200	2571	2938	3257	3489	3591	3698	3796					
7	1	84				851	1027	1144	1318	1491	1663	2059	2452	2840	3225	3551	3984	4170	4274	4386	4490	4586			
7½	1¼	90				859	1050	1240	1366	1554	1741	2110	2537	2959	3318	3735	4143	4442	4579	4704	4802	4913			
8	1¼	96					1152	1300	1522	1668	1887	2320	2749	3173	3664	4081	4494	4905	5210	5345	5470	5607	5717	5920	6104
8½	1¼	102					1243	1414	1586	1840	2009	2511	3006	3416	3906	4386	4789	5264	5738	6035	6166	6311	6447	6678	6903
9	1½	108						1514	1697	1879	2059	2598	3041	3569	4004	4524	5038	5464	5972	6373	6544	6677	6828	7081	7310
9½	1½	114						1618	1827	2034	2240	2753	3259	3761	4358	4852	5342	5830	6313	6890	7284	7446	7597	7902	8154
10	1½	120							1944	2180	2413	2993	3451	4020	4584	5144	5700	6252	6810	7345	7889	8224	8440	8739	9038

Note:—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq. ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \text{Bending Moment for *square slab supported on four sides.}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line. For example showing use of tables see page 69

Table No. 9 **Maximum Stresses: Steel = 20,000 pounds, Concrete = 600 pounds** **Conc. 1:2½:5**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH — CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																						
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	108	162	210	258	275	290	302	315	325	348													
3	¾	36	142	206	277	339	409	439	459	480	496	534	564												
3½	¾	42	171	253	334	415	494	572	638	670	694	747	792	830											
4	¾	48	200	297	411	505	597	690	781	872	911	986	1048	1103	1150										
4½	¾	54	240	344	471	572	697	795	918	1015	1112	1245	1333	1402	1463	1517									
5	1	60		377	508	622	736	849	961	1073	1211	1389	1477	1561	1627	1693	1747								
5½	1	66		421	569	715	824	968	1110	1216	1358	1671	1805	1900	1984	2060	2136	2198							
6	1	72			611	793	928	1063	1239	1371	1502	1892	2141	2256	2370	2461	2544	2628	2698						
6½	1	78			684	850	1014	1177	1338	1499	1659	2080	2445	2645	2775	2882	2990	3078	3169	3253					
7	1	84				946	1142	1271	1464	1657	1848	2289	2724	3061	3209	3324	3463	3574	3663	3759	3848	3931			
7½	1¼	90				955	1168	1379	1519	1728	1935	2346	2821	3272	3415	3566	3703	3810	3927	4034	4120	4215			
8	1¼	96					1280	1445	1690	1853	2097	2577	3054	3525	3902	4061	4207	4341	4465	4581	4689	4806	4900	5075	5232
8½	1¼	102					1383	1571	1762	2045	2232	2789	3341	3796	4338	4564	4715	4880	5032	5174	5284	5409	5526	5724	5917
9	1½	108						1682	1886	2087	2288	2887	3378	3966	4449	4817	5014	5164	5333	5462	5609	5723	5852	6069	6266
9½	1½	114						1798	2030	2260	2490	3058	3620	4179	4842	5362	5558	5741	5910	6099	6243	6381	6511	6773	6989
10	1½	120							2160	2422	2680	3325	3834	4467	5094	5715	6143	6359	6557	6745	6917	7048	7234	7490	7747

Note:—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq. ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \text{Bending Moment for *square slab supported on four sides.}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line.

For examples showing use of tables see page 69

Table No. 10 **Maximum Stresses: Steel = 20,000 pounds, Concrete = 650 pounds** **Conc. { 1:2:4 / 1:2½:5 carefully graded**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH — CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																		
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	108	162	210	263	297	314	328	341	353	377													
3	¾	36	142	206	277	339	409	469	498	520	537	578	611												
3½	¾	42	171	253	334	415	494	572	650	726	752	810	858	900											
4	¾	48	200	297	411	505	597	690	781	872	961	1068	1136	1194	1246										
4½	¾	54	240	344	471	572	697	795	918	1015	1112	1349	1444	1519	1585	1644									
5	1	60		377	508	622	736	849	961	1073	1211	1484	1601	1691	1764	1835	1893								
5½	1	66		421	569	715	824	968	1110	1216	1358	1671	1956	2058	2150	2232	2314	2381							
6	1	72			611	793	928	1063	1239	1371	1502	1892	2234	2444	2568	2666	2756	2848	2922						
6½	1	78			684	850	1014	1177	1338	1499	1659	2080	2445	2857	3008	3123	3240	3334	3431	3525					
7	1	84				946	1142	1271	1464	1657	1848	2289	2724	3156	3476	3601	3752	3872	3968	4072	4169	4259			
7½	1¼	90				955	1168	1379	1519	1728	1935	2346	2821	3290	3688	3863	4012	4128	4255	4371	4464	4566			
8	1¼	96					1280	1445	1690	1853	2097	2577	3054	3525	4071	4400	4558	4703	4837	4963	5080	5207	5309	5498	5668
8½	1¼	102					1383	1571	1762	2045	2232	2789	3341	3796	4338	4874	5108	5286	5452	5605	5725	5860	5987	6201	6410
9	1½	108						1682	1886	2087	2288	2887	3378	3966	4449	5026	5432	5594	5777	5918	6077	6200	6340	6576	6788
9½	1½	114						1798	2030	2260	2490	3058	3620	4179	4842	5391	5936	6220	6403	6607	6764	6914	7055	7538	7571
10	1½	120							2160	2422	2680	3325	3834	4467	5094	5715	6333	6890	7104	7307	7494	7636	7836	8114	8393

Note:—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \text{Bending Moment for *square slab supported on four sides.}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line. For examples showing use of tables see page 69

Table No. 11 **Maximum Stresses: Steel = 20,000 pounds, Concrete = 700 pounds** **Conc. 1:2:4**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH																						
			CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																						
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	108	162	210	263	310	338	353	367	379	406													
3	¾	36	142	206	277	339	409	469	529	560	579	623	658												
3½	¾	42	171	253	334	415	494	572	650	740	810	872	924	969											
4	¾	48	200	297	411	505	597	690	781	872	961	1150	1223	1286	1342										
4½	¾	54	240	344	471	572	697	795	918	1015	1112	1376	1555	1636	1707	1770									
5	1	60		377	508	622	736	849	961	1073	1211	1484	1724	1821	1899	1976	2038								
5½	1	66		421	569	715	824	968	1110	1216	1358	1671	2015	2217	2316	2404	2492	2565							
6	1	72			611	793	928	1063	1239	1371	1502	1892	2234	2572	2765	2871	2968	3066	3147						
6½	1	78			684	850	1014	1177	1338	1499	1659	2080	2445	2857	3240	3363	3489	3591	3698	3796					
7	1	84				946	1142	1271	1464	1657	1848	2289	2724	3156	3583	3878	4040	4170	4274	4386	4490	4586			
7½	1¼	90				955	1168	1379	1519	1728	1935	2346	2821	3290	3688	4150	4320	4442	4579	4704	4802	4913			
8	1¼	96					1280	1445	1690	1853	2097	2577	3054	3525	4071	4534	4908	5065	5210	5345	5470	5607	5717	5920	6104
8½	1¼	102					1383	1571	1762	2045	2232	2789	3341	3796	4338	4874	5320	5693	5871	6035	6166	6311	6447	6678	6903
9	1½	108						1682	1886	2087	2288	2887	3378	3966	4449	5026	5598	6024	6222	6373	6544	6677	6828	7081	7310
9½	1½	114						1798	2030	2260	2490	3058	3620	4179	4842	5391	5936	6477	6895	7115	7284	7446	7597	7902	8154
10	1½	120							2160	2422	2680	3325	3834	4467	5094	5715	6333	6946	7555	7869	8071	8224	8440	8739	9038

Note:—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

Area of Steel Required per Foot of Width for a Maximum Resisting Moment of Slab of Given Thickness

Corresponding SAFE BENDING MOMENT due to applied load and weight of floor:

$$M = \frac{wl^2}{10} = \frac{1}{10} \times \text{Load per sq. ft.} \times (\text{length of span})^2 = \text{Bending Moment for slab supported on two sides.}$$

$$M = \frac{wl^2}{20} = \frac{1}{10} \times \text{Bending Moment for *square slab supported on four sides.}$$

*For a slab that is not square supported on four sides see page 71

The Maximum Allowable Fiber Stress in the steel governs the values of Resisting Moments given below and to the left of the heavy zigzag line; the Maximum Allowable Fiber Stress in the concrete governs the values above and to the right of this line.

For examples showing use of tables see page 69

Table No. 12 **Maximum Stresses: Steel = 20,000 pounds, Concrete = 750 pounds** **Carefully graded 1:2:4 Conc.**

Total Thickness of Slab Inches	Center of Steel to Bot. of Slab	Weight of Slab per sq. ft. Pounds	MOMENTS OF RESISTANCE IN FOOT POUNDS PER FOOT OF WIDTH																						
			CROSS SECTIONAL AREA IN SQUARE INCHES OF STEEL REINFORCEMENT PER FOOT OF WIDTH																						
			.04	.06	.08	.10	.12	.14	.16	.18	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.00
2½	¾	30	108	162	210	263	310	361	378	394	406	435													
3	¾	36	142	206	277	339	409	469	529	597	620	667	704												
3½	¾	42	171	253	334	415	494	572	650	740	816	934	990	1038											
4	¾	48	200	297	411	505	597	690	781	872	961	1193	1311	1378	1438										
4½	¾	54	240	344	471	572	697	795	918	1015	1112	1376	1658	1753	1829	1896									
5	1	60		377	508	622	736	849	961	1073	1211	1484	1754	1951	2035	2117	2184								
5½	1	66		421	569	715	824	968	1110	1216	1358	1671	2015	2321	2481	2576	2672	2748							
6	1	72			611	793	928	1063	1239	1371	1502	1892	2234	2572	2949	3076	3180	3286	3372						
6½	1	78			684	850	1014	1177	1338	1499	1659	2080	2445	2857	3265	3603	3739	3845	3962	4067					
7	1	84				946	1142	1271	1464	1657	1848	2289	2724	3156	3583	3946	4329	4468	4579	4699	4810	4914			
7½	1¼	90				955	1168	1379	1519	1728	1935	2346	2821	3290	3688	4150	4608	4763	4909	5043	5149	5368			
8	1¼	96					1280	1445	1690	1853	2097	2577	3054	3525	4071	4534	4994	5427	5582	5726	5860	6006	6126	6343	6540
8½	1¼	102					1383	1571	1762	2045	2232	2789	3341	3796	4338	4874	5320	5849	6291	6468	6606	6762	6907	7155	7396
9	1½	108						1682	1886	2087	2288	2887	3378	3966	4449	5026	5598	6071	6635	6828	7014	7154	7318	7587	7832
9½	1½	114						1798	2030	2260	2490	3058	3620	4179	4842	5391	5936	6477	7015	7623	7805	7980	8140	8467	8738
10	1½	120							2160	2422	2680	3325	3834	4467	5094	5715	6333	6946	7555	8161	8648	8811	9042	9363	9684

Note :—For sectional areas, weights, etc., of Triangle Mesh Wire Reinforcement see pp. 110-111.

TABLE No. 13.

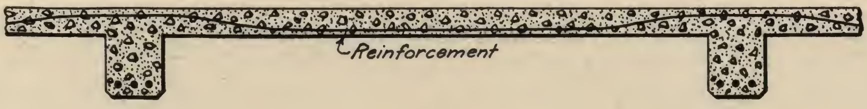
RECOMMENDED MINIMUM DEPTH OF SLABS IN INCHES.

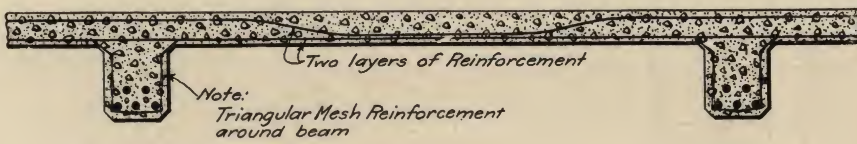
SPAN IN FEET. MIXTURE 1:2:4.

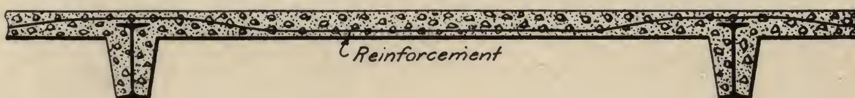
Live load in lbs. per sq.ft.	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
50	2.5	2.5	3.0	3.0	3.5	3.5	4.0	4.0	4.5	5.0	5.0	5.5	5.5	6.0	6.5	7.0	7.5
100	3.0	3.0	3.0	3.5	3.5	4.0	4.5	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0
150	3.0	3.0	3.5	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0
200	3.5	3.5	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.5	11.0
250	3.5	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.5	10.0	10.5	11.0	12.0
300	4.0	4.0	4.0	4.5	5.0	5.5	6.5	7.0	7.5	8.0	8.5	9.5	10.0	10.5	11.5	12.0	
350	4.0	4.0	4.5	5.0	5.5	6.0	6.5	7.5	8.0	8.5	9.0	10.0	10.5	11.5	12.0		
400	4.5	4.5	4.5	5.0	5.5	6.5	7.0	7.5	8.5	9.0	9.5	10.5	11.0	12.0			
450	4.5	4.5	4.5	5.5	6.0	6.5	7.5	8.0	8.5	9.5	10.0	11.0	11.5				
500	4.5	4.5	5.0	5.5	6.0	7.0	7.5	8.5	9.0	10.0	10.5	11.5	12.0				

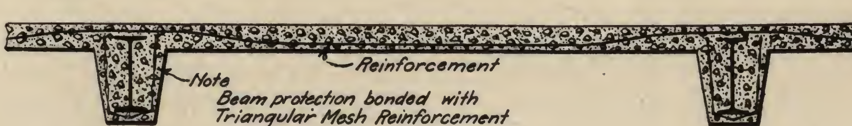
The depths given are the total thickness of the slab, assuming the center of the reinforcement to be ¾ inch above bottom. More covering than this may be used by increasing the depth, the extra weight of this concrete being added to the live load.

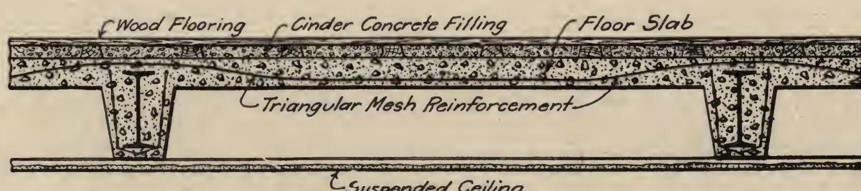
It is always allowable to use depths greater than those here specified, thereby decreasing the amount of reinforcement and increasing the amount of concrete. It is also possible to use somewhat less depths, but not economical. For depths greater than 12 inches, it is more economical to use reinforced concrete beams with thinner slabs between. These depths may also be used for a carefully graded mixture of 1:2½:5 concrete.


Reinforcement


Two layers of Reinforcement
Note:
Triangular Mesh Reinforcement
around beam


Reinforcement


Reinforcement
Note
Beam protection bonded with
Triangular Mesh Reinforcement


Wood Flooring
Cinder Concrete Filling
Floor Slab
Triangular Mesh Reinforcement
Suspended Ceiling

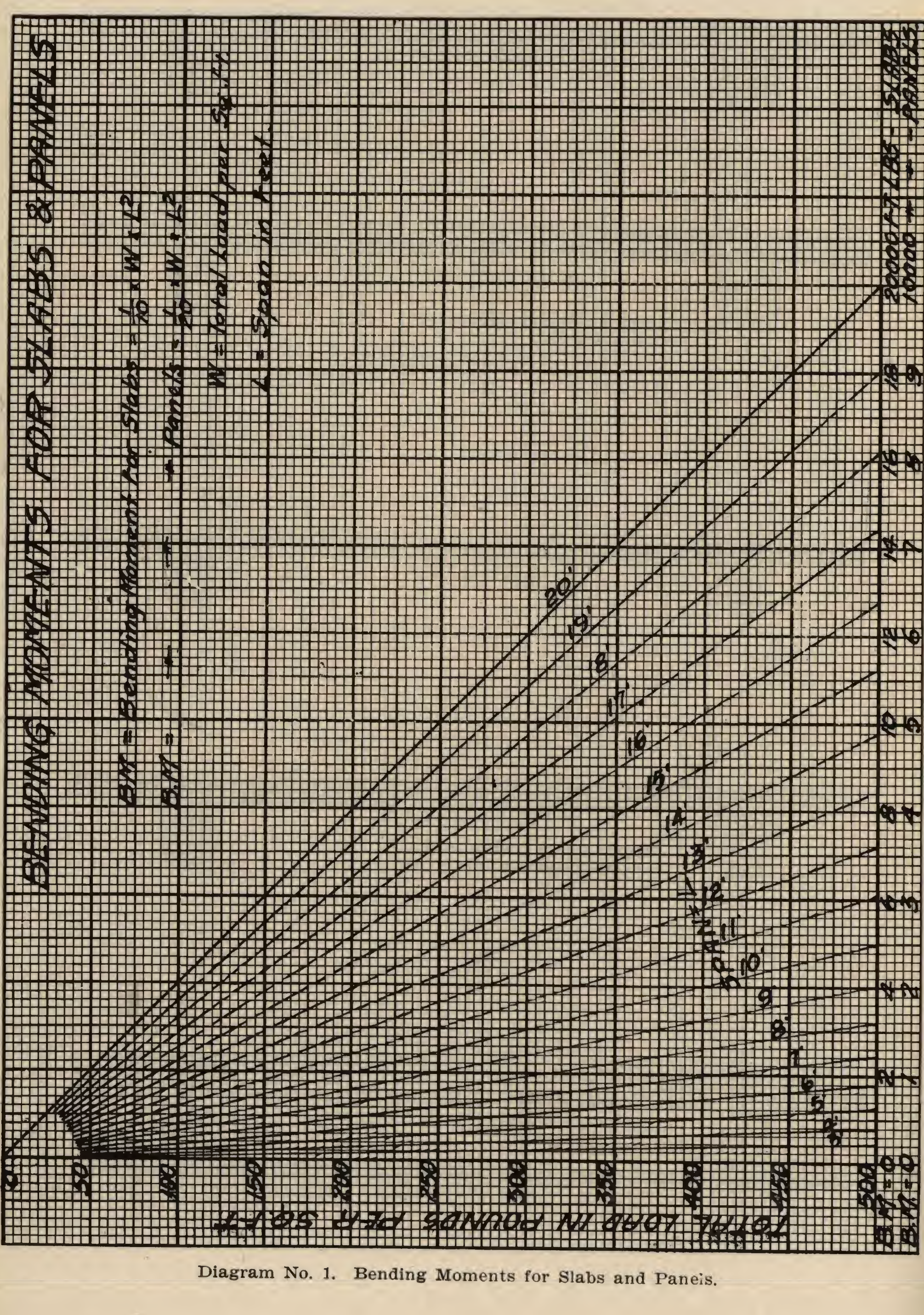


Diagram No. 1. Bending Moments for Slabs and Panels.

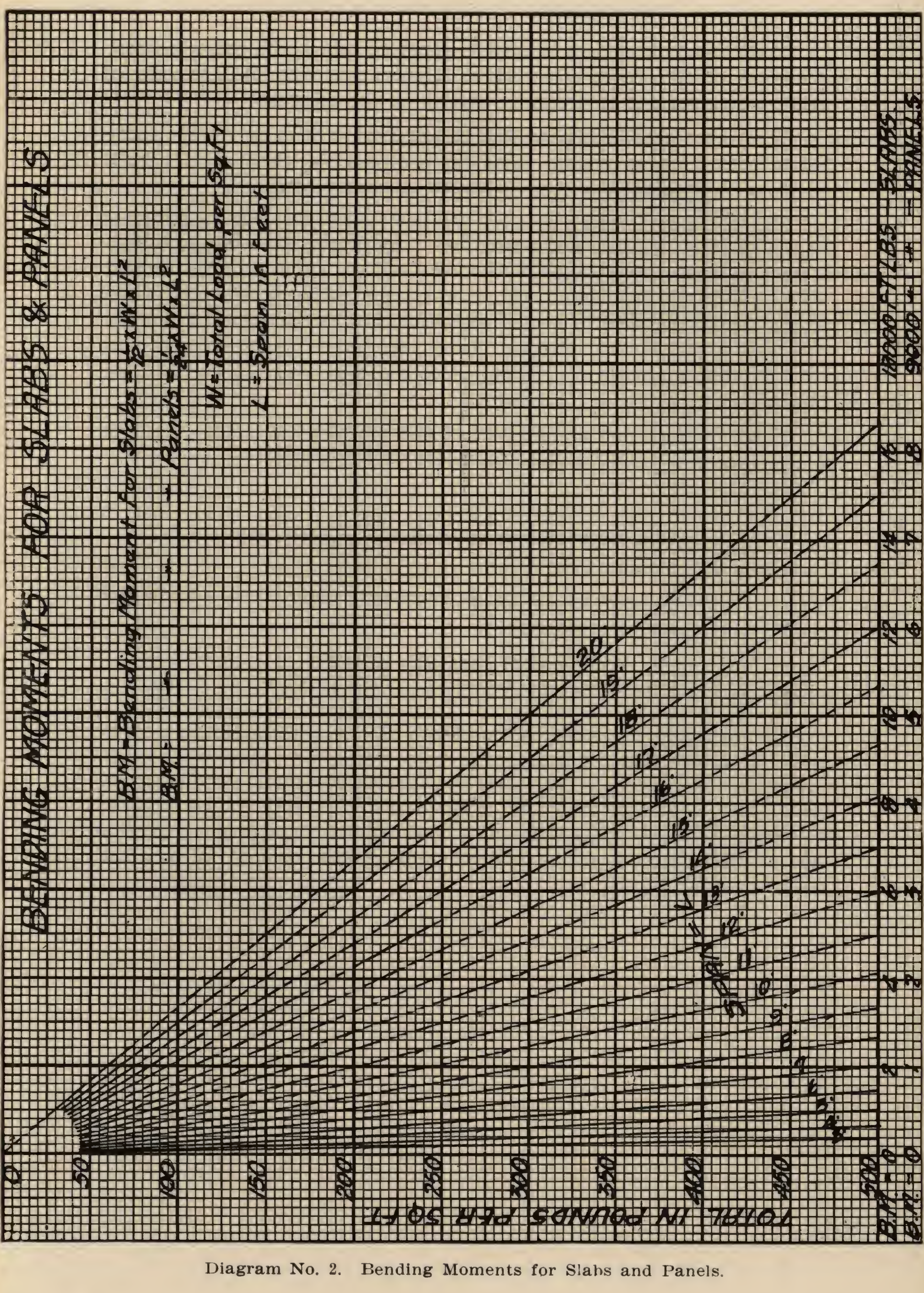


Diagram No. 2. Bending Moments for Slabs and Panels.

REINFORCED CONCRETE COLUMNS.

Vertical reinforcement is used in concrete columns in order to carry a portion of the direct compressive stresses and also to take care of bending stresses due to eccentric loading on the columns.

If bars are used, it is necessary to band or tie them together. These bands should be so spaced that the unsupported length of the rods between bands is not too great to permit of column action.

The column sketches on page 91 show how **Triangle Mesh Wire Fabric** is used in column construction. The fabric is constructed with a joint at each longitudinal so that the column cages may be formed without bending the wires. The cross wires act as bands holding the longitudinal members in place and also add a factor of safety to the construction by reason of their "hoop" action. If it is necessary to have a greater section of vertical steel, place bars inside of the wire fabric.

The size of column and amount of reinforcement necessary to carry a given load may be determined by means of the formula:—

$$P = A_c f_c + n f_c A_s$$

Where P = Load on column in pounds.

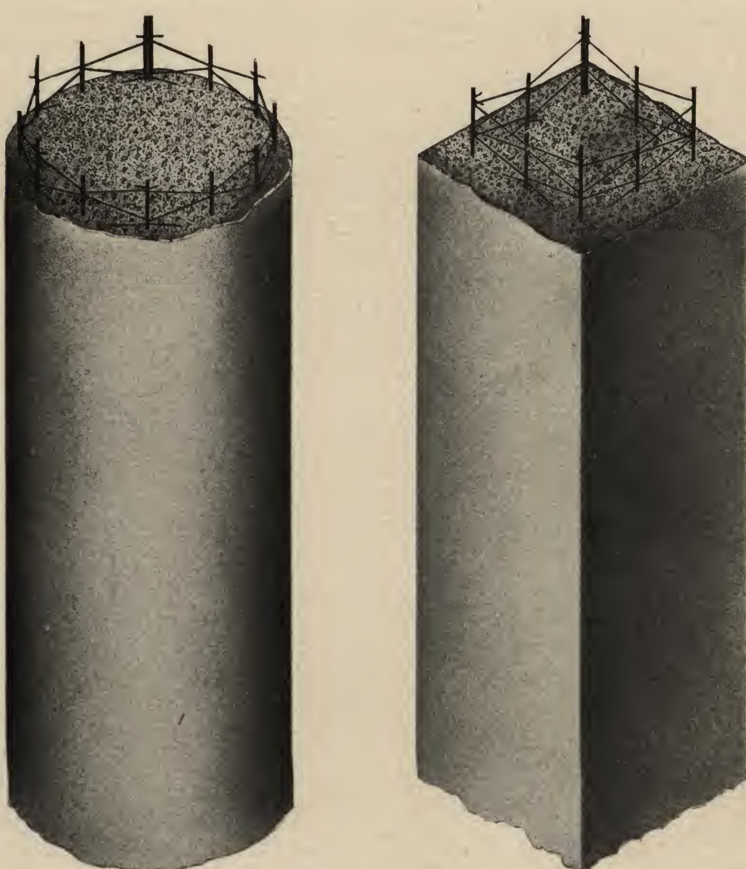
A_c = Cross sectional area of concrete, square inches.

A_s = Cross sectional area of steel, square inches.

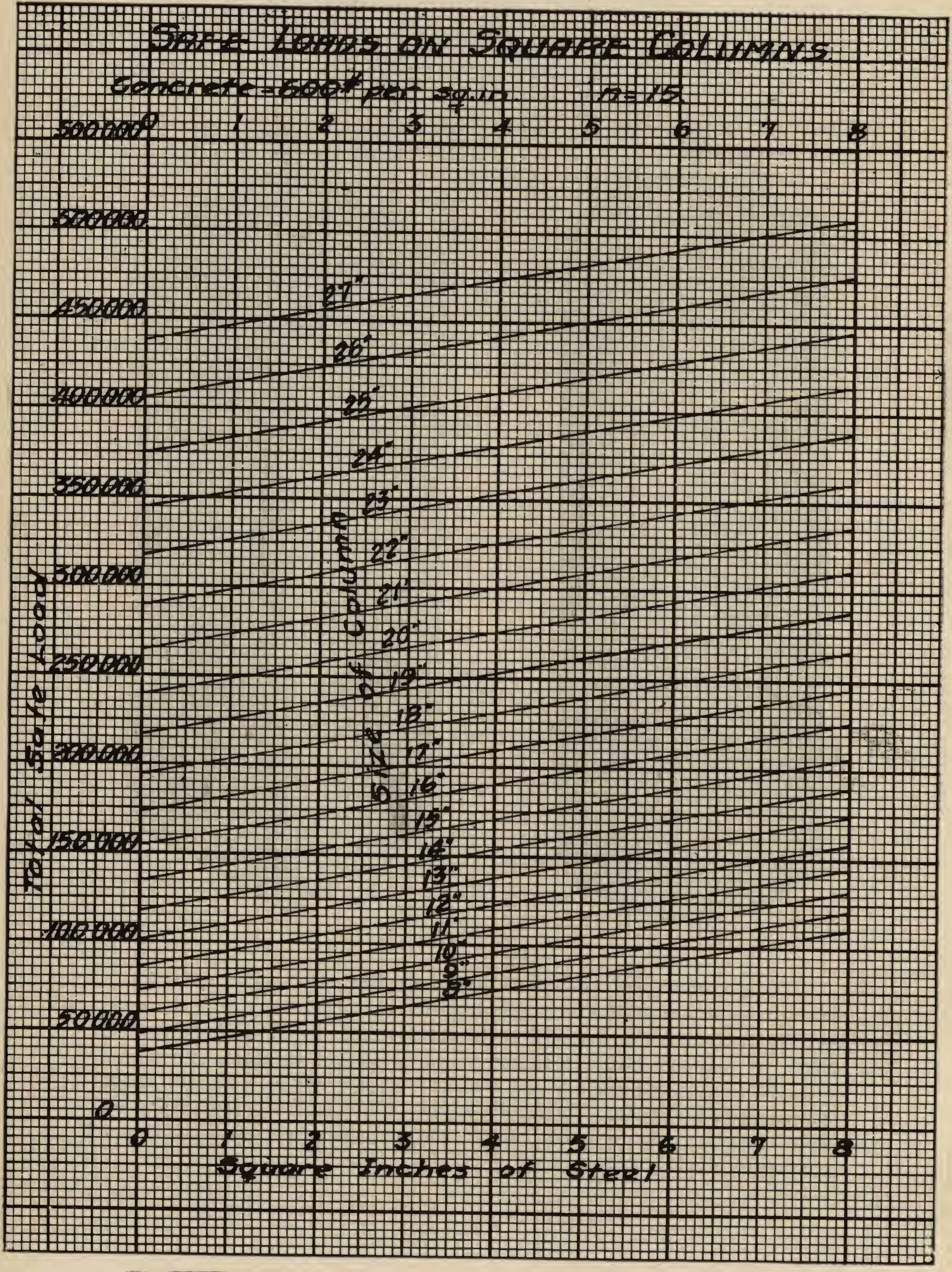
f_c = Allowable stress in concrete in compression, pounds per square inch.

$$n = \text{ratio, } \frac{\text{Modulous of Elasticity of Steel}}{\text{Modulous of Elasticity of Concrete}}$$

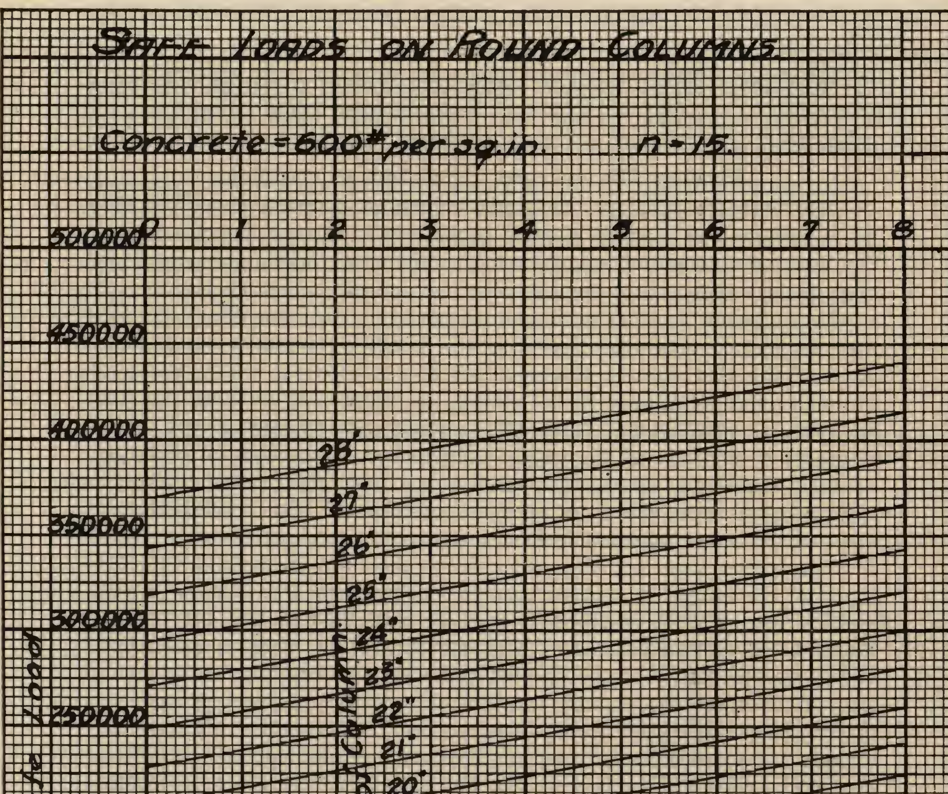
The following diagrams, pages 92 and 93, for Safe Loads on Square and Round Columns are based on a value of 15 for n and 600 pounds per square inch for f_c. They show the amount of vertical reinforcement required in order that a column of given size will carry the given load. An examination of these diagrams shows that with a given load, the size of column may be varied by varying the amount of vertical reinforcement.



Triangle Mesh Reinforcement is most applicable to columns for hoop reinforcement, due to the hinge joint provided at each longitudinal member, thus allowing the material to be readily folded without bending the wires. It may be used for either round or square columns of various diameters, due to the number of different widths in which **Triangle Mesh Reinforcement is made.** The accompanying illustrations demonstrate its adaptability for column work. Additional bars may be included for vertical reinforcement if so desired.



Safe Loads and Area in Square Inches of Steel for Square Columns.



Safe Loads and Area in Square Inches of Steel for Round Columns.

MECHANICS OF PIPES AND RINGS SUBJECT TO EXTERNAL PRESSURE.

Note—Through the courtesy extended by Arthur N. Talbot, University of Illinois, we reprint the following extracts from their Bulletin No. 22, dated April 29th, 1908, on reinforced concrete culvert pipe, sewer pipe, etc.

Bending Moment and Conditions of Loading.

The stresses developed in rings subject to external earth pressure, as in sewers and railroad culvert pipes, are of course dependent upon the bending moments developed, and, as the exact load coming upon the ring and its distribution over the surface are difficult to determine, the bending moment is in general quite uncertain. The amount of the load and its distribution, and therefore the bending moments on different parts of the ring, depend upon a number of conditions, among them the nature of the earth used in the filling, the method of bedding the pipe, the way of tamping the earth at the sides, the amount of the lateral restraint or pressure of the earth horizontally, the method of filling and packing the earth above, the condition of moisture in the earth, etc. Evidently in such earth as saturated quick-sand, the conditions may approach those of external hydrostatic pressure, and on the other hand, in deep sewer trenches, the earth filling may act in such a way that much of its weight is carried against the sides of the trench. In discussing the stresses in rings, it may be well first to find the bending moment for certain assumed conditions of loading, then to make tests under various conditions of loading, and finally to compare these results with a view of determining the probable range of bending moments under the actual conditions of construction. The assumed loadings may include (1) a concentrated load at the crown of the ring, (2) a vertical load distributed uniformly over the horizontal section, (3) a distributed vertical load together with a horizontal load distributed vertically over the sides of the ring, and (4) an oblique loading. In these calculations, since much uncertainty is involved, the difference in the intensity of the load at the crown and at the extremities of the horizontal diameter, due to the different depths of earth, need not be considered. In general the pressures and distribution on the lower half of the ring will be considered to be the same as on the upper half. It is apparent that in a ring of considerable thickness in comparison with its diameter there is a different distribution of stresses from that found in thin rings, but for the rings under consideration the simplicity of analysis for thin rings will outweigh the small loss in accuracy. The possible modifications and complications in the analysis of thick rings may also be considered. As refinements are not essential and approximations are permissible, the analysis will assume a thin ring of homogeneous material having a constant modulus of elasticity and it will also be assumed that the changes from a circular form will have little effect upon the dimensions of the ring.

Concentrated Vertical Load on Thin Elastic Ring.

Consider that a concentrated load Q is applied along the top element of a cylindrical ring and that the ring is supported along an element at the bottom, as indicated in Fig. 1 (a). Since the ring is a continuous curved beam, the analysis will require a slight modification of the convention commonly used for simple straight beams. However, in any segment of the ring, the external forces acting on the ring will be held in equilibrium by the internal or resisting forces acting upon this segment at its two ends. The moment of the internal forces acting at right angles to a section of the ring at an end of the segment is the resisting moment developed, and the bending moment may be considered to be an equal moment having the opposite sign. If we take a quadrant of the ring, as shown in Fig. 1 (b), it is evident from a consideration of the external and internal forces acting upon this ring that this quadrant will be in equilibrium under the action of $\frac{1}{2}Q$ at B, a reaction or thrust of $\frac{1}{2}Q$ at A, a resisting moment in the section of the ring at A which we will call M_A, and a resisting moment in the section of the ring at B which we will call M_B. The amounts of the two resisting moments so developed and thus of the two bending moments it is important to determine. Similarly, if we consider a portion of the ring shown in Fig.

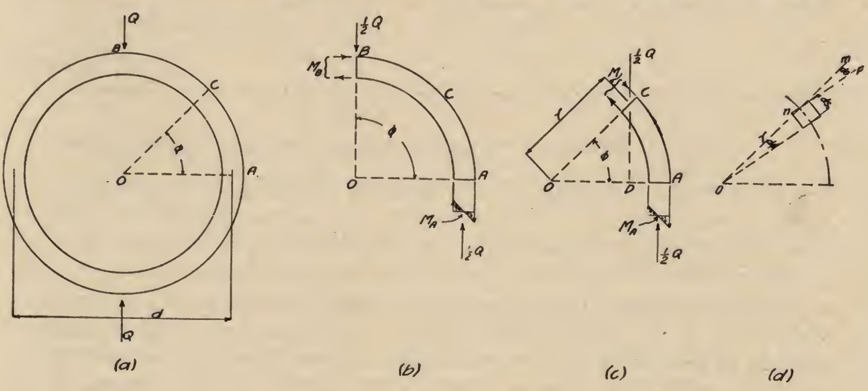


Fig. 1. Ring Under Concentrated Load.

1 (c), the forces which hold it in equilibrium may be shown to be $\frac{1}{2}Q$ at A, $\frac{1}{2}Q$ at C, the moment M_A at the section A, and a variable moment M at C, the value of which will change with a change in the angle ϕ. Taking moments about A, the following equation for the value of the bending moment at any point on the ring results:

$$M = \tfrac{1}{2}Qr\,(1 - \cos\phi) - M_A \ldots\ldots\ldots\ldots\ldots (1)$$

$$\text{*but, } M_A = \frac{Qr}{2}\left(1 - \frac{1}{\frac{1}{2}\pi}\right) = .091\,Qd \ldots\ldots\ldots\ldots\ldots (4)$$

where d is the mean diameter of the ring.

* See Bulletin No. 22, University of Illinois, for full mathematical proof.

Substituting this value of M_A in equation (1) and making $\phi = 90°$, we have

$$M_B = \frac{Qd}{4} - .091\,Qd = .159\,Qd \ldots\ldots\ldots\ldots\ldots (5)$$

It will be seen that the bending moment at B is about sixteen-ninths times that at A.

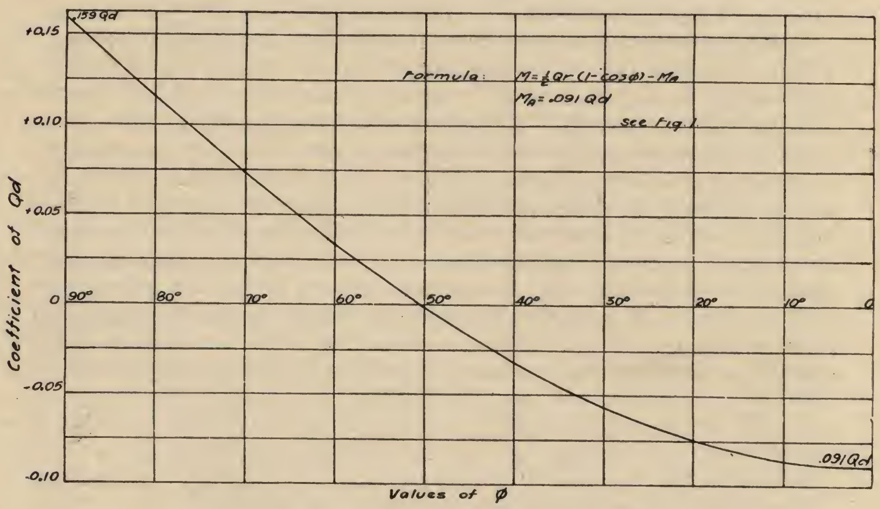


Fig. 2. Variation in Bending Moment for Concentrated Load.

To determine the point of zero bending moment place equation (1) equal to zero. $\cos\phi = .636$ and $\phi = 50°\ 30'$. At this point the algebraic sign of the bending moment changes from negative to positive.

Fig. 2 gives the variation of the bending moment from A to B.

It may be shown that if the load be applied equally at two points on either side of the crown (and similarly supported below) the bending moment at the crown will be decreased and that if these points are immediately above the quarter points of the diameter the value of the bending moment at the crown becomes 0.054 Qd and that at the extremities of the horizontal diameter 0.071 Qd. This has a bearing upon the effect of the methods of bedding a pipe.

Distributed Vertical Load on Thin Elastic Ring.

Consider that the vertical load is distributed uniformly over the horizontal projection of the ring, as shown in Fig. 3 (a), and call w the load per lineal unit of horizontal width for a ring one unit long and r the mean radius of the ring.

The expression for the bending moment M at any point C on the ring is found by taking moments about C.

$$M = wr^2 (1 - \cos\phi) - \tfrac{1}{2} wr^2 (1 - \cos\phi)^2 - M_A$$
$$= \tfrac{1}{2} wr^2 (1 - \cos^2\phi) - M_A \quad \ldots\ldots\ldots\ldots (6)$$

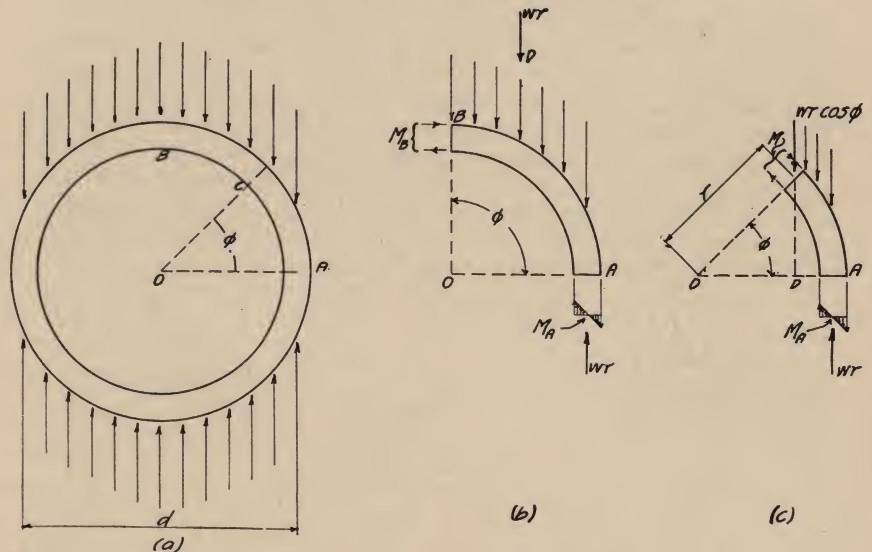


Fig 3 Ring Under Distributed Vertical Load.

*But, $-M_A = M_B = \frac{1}{16} wd^2 = \frac{1}{16} Wd$ .... (7)
where d is the mean diameter of the ring and W is the total load on a ring of unit length.

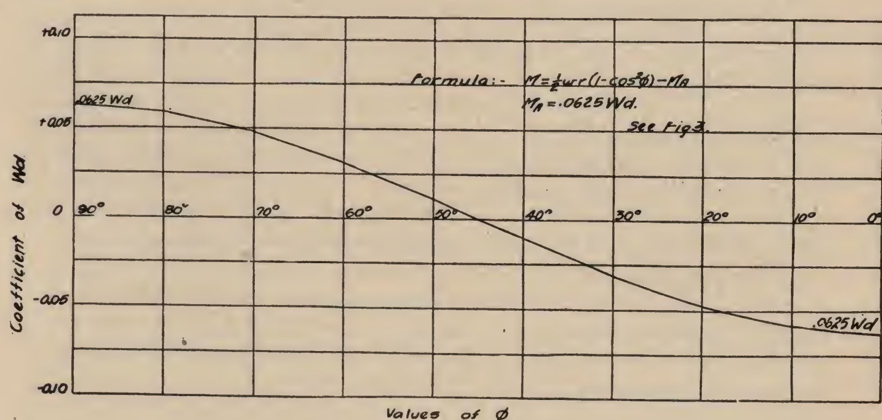


Fig. 4. Variation in Bending Moment For Distributed Load.

* See Bulletin No. 22, University of Illinois, for full mathematical proof.

Distributed Vertical and Horizontal Loads on Thin Elastic Ring.

Let us consider that the vertical load is distributed over the horizontal section of the pipe as before (w per lineal unit of width of pipe) and that there is a horizontal pressure uniformly distributed vertically against the pipe, the amount of this horizontal pressure per lineal unit of vertical distance being qw, where q is the ratio of the horizontal to the vertical intensity of pressure. The conditions are indicated in Fig. 5 (a). We may consider that the effect of these loads is the combined effect of the two loads. Call M', M'_A, and M'_B the bending moments produced by the vertical load and M'', M''_A and M''_B, the bending moments produced by the horizontal load. The bending moment at any section C (Fig. 5 (b)) produced by the vertical load is

$$M' = wr^2 (1 - \cos \phi) - \tfrac{1}{2} wr^2 (1 - \cos \phi)^2 - M'_A.$$

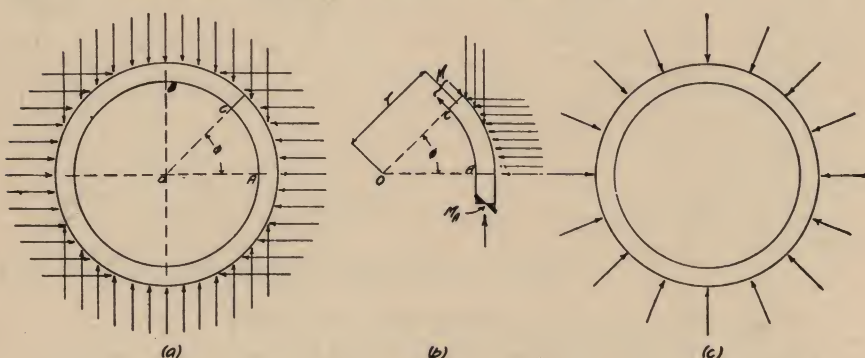


Fig. 5. Ring Under Distributed Vertical and Horizontal Load.

It may be shown that the bending moment produced at any section C by the horizontal load is

$$M'' = -\tfrac{1}{2} qwr^2 \sin^2 \phi + M''_A$$

and that the value of M''_A is $\tfrac{1}{4}$ qwr^2. The resulting moment therefore is

$$M = M' + M'' = \tfrac{1}{4} wr^2 [1 + q - 2 \cos^2 \phi - 2q \sin^2 \phi] \; \ldots\ldots (8)$$

The moment at B and A will therefore be

$$M_B = - M_A = \tfrac{1}{4} (1 - q) wr^2 = \tfrac{1}{16} (1-q) \; Wd \ldots\ldots (9)$$

where W is the total vertical load on the ring. The bending moment becomes zero at $\phi = 45°$ as in the other case.

If the intensity of the horizontal pressure is the same as that of the vertical pressure, $q = 1$ and M becomes zero at all points. This corresponds to uniform external pressure as shown in Fig. 5 (c) and produces equal compression in all parts of the ring.

Resisting Moment and Calculation of Stresses.

For a ring whose thickness is small in comparison with the diameter the difference in the length of the inner fiber and outer fiber is small and

the expression for the resisting moment given for ordinary straight beams may be applied with a close degree of approximation.

For a ring made of reinforced concrete the conditions differ somewhat from those o. a homogeneous elastic material. For ordinary cases it will be not far from the truth to equate the bending moment determined as above and the resisting moment of the reinforced concrete section. As the amount of reinforcement is usually lower than that in which the circular beam would fail by compression in the concrete, we may, without material error, take for the resisting moment of the reinforced concrete section the value .87 Aft, where (t) is the distance from the compression face to the center of the steel reinforcement, A is the area of the cross-section of the reinforcement for a unit of length of ring, and f is the tensile unit-stress in the steel due to the bending moment. To equate the bending moment determined as before to this resisting moment is not exactly correct, since among other reasons the neutral axis does not come at the center of the thickness of the ring (which is the point about which the bending moments were taken), and since the elastic curve is not the same as in a ring of homogeneous material, and hence the distribution and amounts of the bending moments will not be exactly the same. However, the use of the bending moments determined for homogeneous rings is the nearest approximation we have, and is not seriously in error. At sections where thrust occurs, as at A, (Fig. 3), the tension in the steel determined as above will be reduced by the resisting compressive stresses there set up. The amount of the tension in the steel at the point A may be calculated by the formula

$$f' = f - \frac{\frac{1}{2}nT}{t(1+np)} \quad \text{..................} (19)$$

which is applicable for both concentrated and distributed loads. In this formula f is the tensile stress in the steel due to the bending moment (as calculated by equating .87 Aft to the bending moment at the section considered), p is the ratio of the area of reinforcement for a unit length of beam or ring to the distance between the center of the steel and the compression face of the concrete, T is the thrust or pressure against the face of the section, and n is the ratio of the moduli of elasticity of steel and concrete, which, for purposes of this calculation, may be taken as 15. At the extremity of the horizontal diameter the thrust is $\frac{1}{2}$ W. At the crown it is zero for vertical loading, and for both concentrated and distributed load the greatest tensile stress is found at this section.

Conditions of Bedding and Loading Found in Practice.

The foregoing discussion assumes certain definite conditions of loading. These are useful in establishing definite formulas which may be used as a basis for calculations. It is not to be expected that these conditions represent accurately the condition of bedding and loading to be found in practice. It is then desirable that the nature and extent

of possible or probable variations from these assumed conditions be discussed and the effects of such a divergence considered. The following are suggestions of variations; the engineer will easily extend the discussion by numerous examples taken from his own experience.

If the layer of earth immediately under the pipe is hard or uneven, or if the bedding of the pipe at either side is soft material or not well tamped, the main bearing of the pipe may be along an element at the bottom and the result is in effect concentrated loading. The result is to greatly increase the bending moment developed and hence the tendency of the pipe to fail. This condition may be aggravated in the case of a pipe with a stiff hub or bell where settlement may bring an unusual proportion of the bearing at the bell and the distribution of the pressure be far from the assumed condition. In bedding the pipe in hard ground it is much better to form the trench so that the pipe will surely be free along the bottom element, even after settlement occurs, and so that the bearing pressures may tend to concentrate at points say under the one-third points of the horizontal diameter (or even the outer quarter points). This will reduce the bending moments developed in the ring.

In case the pipe is bedded in loose material, the effect of the settlement will be to compress the earth immediately under the bottom of the pipe more completely than will be the effect at one side, with the result that the pressure will not be uniformly distributed horizontally. Similarly, in a sewer trench, if loose material is left at the sides and the material at the extremity of the horizontal diameter is loose and offers little restraint, the pressure on the earth will not be distributed horizontally and the amount of bending moment will be materially different from that where careful bedding and tamping give an even distribution of bearing pressure over the bottom of the sewer.

In case of a small sewer in a deep trench, the load upon the sewer may be materially less than the weight of the earth above, where the earth forms a hard compact mass and is held by pressure and friction against the sides of the trench.

In case a culvert pipe is laid in an ordinary embankment by cutting down the sides slopingly, it is evident that the load which comes upon the pipe will be materially less than the weight of the earth immediately above it. If a culvert pipe replaces a trestle and the filling is allowed to run down the slope, the direction and amount of the pressure against the pipe will differ considerably from that which obtains in a trench or in the case of a level filling. It is possible in the latter case that the smaller amount of settlement of the earth directly over the culvert pipe, due to the greater depth of earth on the adjacent sections, may allow a greater proportion of the load to rest upon the culvert pipe than would ordinarily be assumed.

Attention should be called to the fact that the distribution of the pressure by means of earth under and over a ring assumes that the

earth is compressed in somewhat the same way as when other material of construction is given compression. Unless the earth has elasticity, the distribution of pressure cannot occur. To secure the uniform distribution assumed the ring itself must give enough to allow for the movement of the earth which takes place under pressure. This is especially true with reference to the presence and utilization of lateral restraint, and a ring which does not give laterally, as for example a plain concrete ring, will not develop lateral pressure in the adjoining earth under ordinary conditions of moisture and filling to any great extent. As the conditions of earth and moisture produce mobility and approach hydrostatic conditions, the necessity for this elasticity and movement do not exist, but here the lateral pressure approaches the vertical pressure in amount and the bending moments become relatively smaller.

The discussion is sufficiently extended to indicate the importance of care in bedding culvert pipe and sewers and in filling over them, and to indicate the great difference in the amount of bending moment developed with different conditions of bedding and filling. Where there is any question of needed strength, it will be money well expended to use care and precaution in bedding the pipe and in filling around and over it. I am convinced that a little extra expense will add considerable stability, life, strength, and safety to such structures, far out of proportion to the added cost. It is possible that under careful conditions of laying, lighter structures may be used with a saving in the cost of construction.

Summary.

From the tests and the discussions it would seem evident that among the facts brought out are the following:

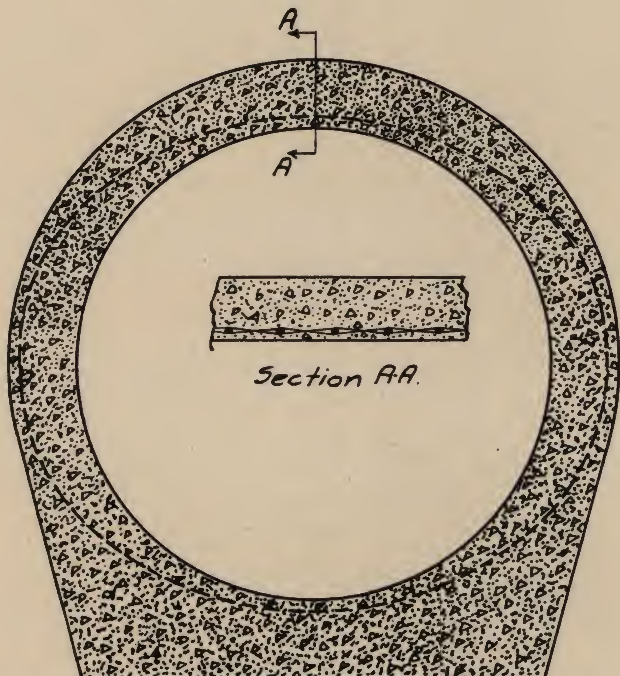
The reinforced concrete rings in the concentrated load tests held their maximum loads or about their maximum loads through a considerable deflection, thus showing a quality which is of value when changes in earth conditions permit a gradual yielding of the surrounding earth. The calculated restraining moment agrees fairly well with the calculated bending moment.

The reinforced concrete rings and pipes tested under distributed load made a satsfactory showing. The so-called critical failure may occur by either tension failure in the steel or a diagonal tension failure (ordinarily called shearing failures) in the concrete. A flattened arc for the reinforcement where it approaches the inner face is of assistance and stirrups may be of some value. Beyond the critical load the reinforcement is of service in distributing the cracks and in holding the concrete together. Final failure is by crushing of the concrete in much the same way as was obtained with the plain concrete rings. The additional strength beyond the critical load may be taken into consideration in selecting the factor of safety or working strength.

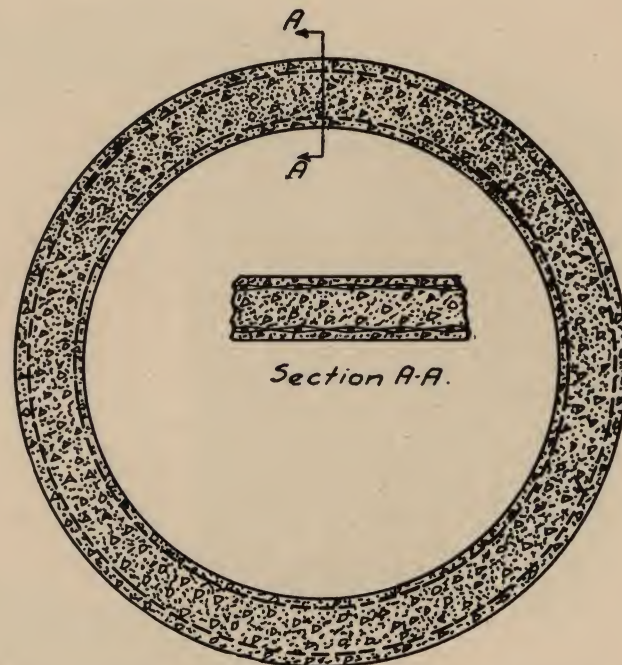
The restraint of the sand in the tests is very important, and the effect is to reduce the bending moment developed by a given vertical

load, or, as it would be commonly stated, to add strength to the pipe. The degree of permanency of this side restraint is uncertain. It seems evident in these tests that the distribution of the pressure, both horizontal and vertical, was not uniform, and that with the usual method of placing a pipe in an embankment, and especially when other materials than sand are used, the distribution would be even less uniform than here found. In view of this it will be well in making calculations and designs to use the formula $\frac{1}{16}$ Wd for the bending moment, thus considering that the side of restraint is offset by the uneven distribution of the load, any surplus from this being considered merely an additional margin of safety. For pipes poorly bedded and filled a larger bending moment than $\frac{1}{16}$ Wd should be used.

The method of bedding and laying pipes and the nature of the bed and the surrounding earth have a great effect upon the bending moment developed and upon the resistance of the pipe to failure. If the method of laying, or the hardness of the soil below, or the condition of the settlement of the pipe is such that the pipe is supported only or mainly along an element of the cylinder at the bottom the bending moment developed will be greatly increased over that of a uniformly distributed support. If the greatest supported pressure comes at points well to the side of this bottom element, as may be obtained by careful bedding, the bending moment is reduced. It is also plain that the bell should be left free from pressure at the bottom. It is possible that the presence of the bell detracts from the strength of the pipe. Any action in filling which increases the lateral restraint against the pipe will add to the security of the structure.

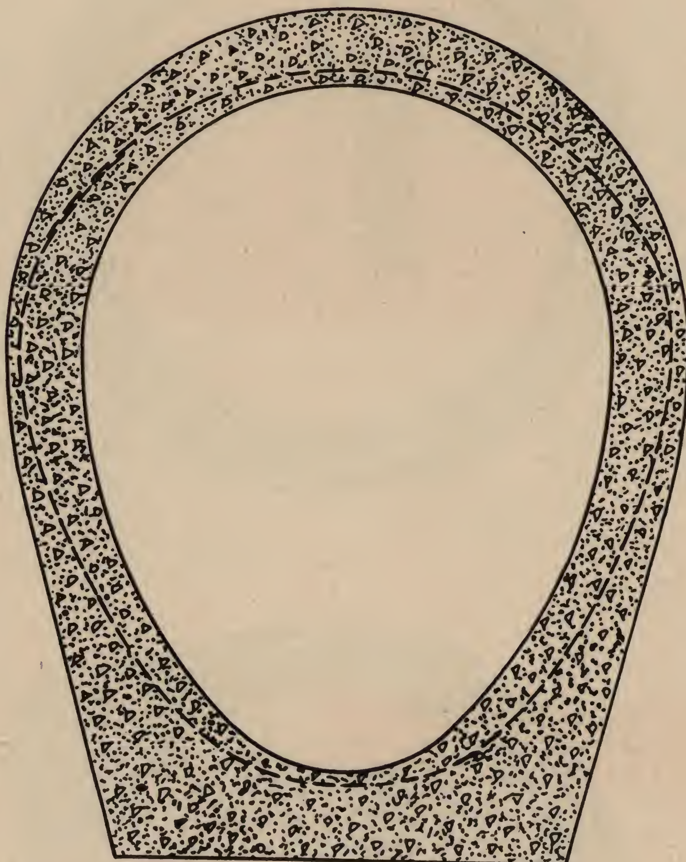


Typical Sewer Section Reinforced with One Layer of Triangle Mesh Reinforcement.

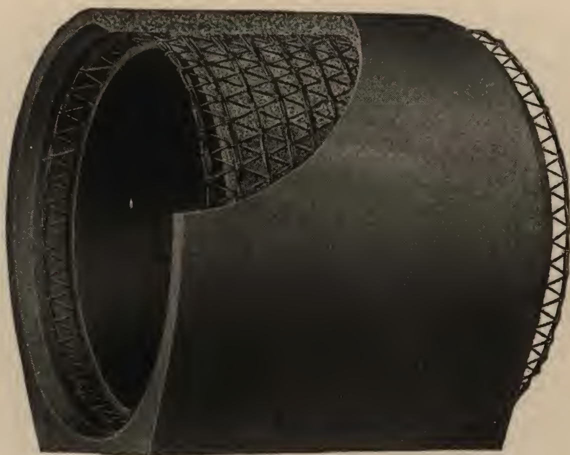


Typical Sewer Section Reinforced with Two Layers of Triangle Mesh Reinforcement.

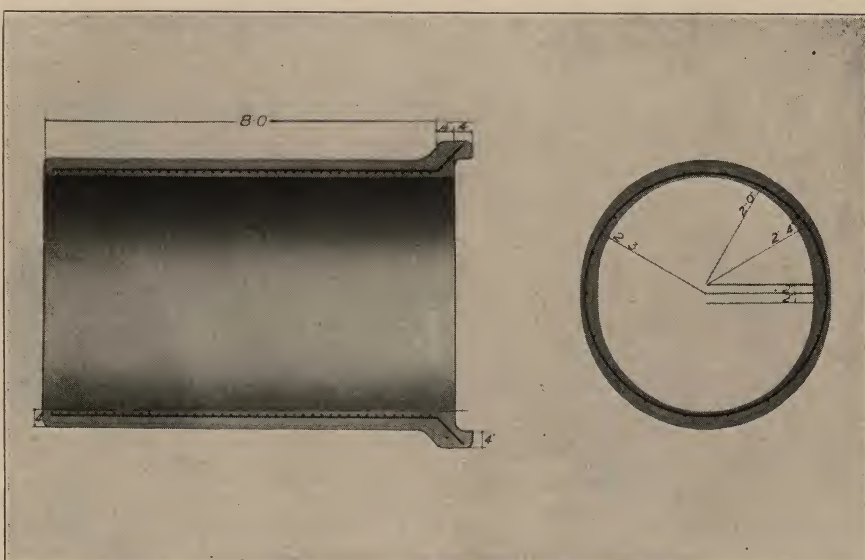
Note Continuous Reinforcement.



Typical Sewer Section Reinforced with One Layer Triangle Mesh Reinforcement. Note Continuous Reinforcement.



Triangle Mesh Reinforced Concrete Sewer Pipe as Made by the Lock Joint Pipe Company.



Triangle Mesh Wire Reinforced Concrete Pipe.
As made by the American Concrete Company.

TRIANGLE MESH WIRE REINFORCED CONCRETE PIPE.
As Made by Reinforced Concrete Pipe Co.

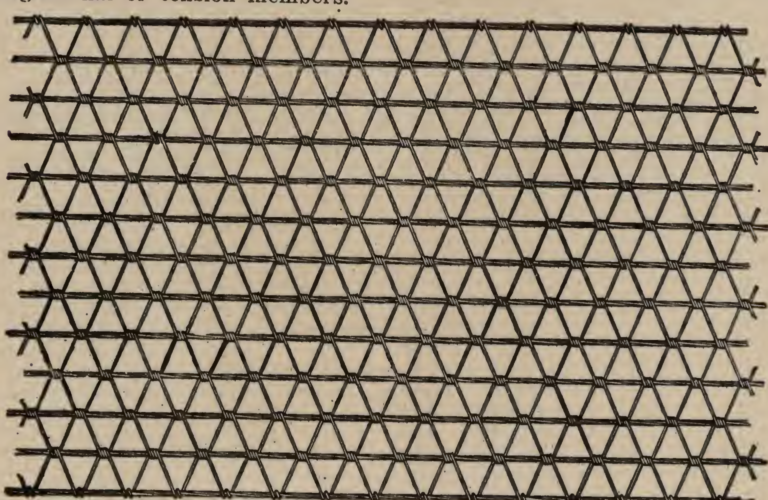
DOUBLE TRIANGLE WIRE MESH REINFORCED CONCRETE PIPE.
As Made by Reinforced Concrete Pipe Co.

TRIANGLE MESH STEEL WIRE REINFORCEMENT.

Triangle Mesh Steel Woven Wire Reinforcement is made with both single and stranded longitudinal, or tension members. That with the single wire longitudinal is made with one wire varying in size from a No. 12 gauge up to and including a ½-inch diameter, and that with the stranded longitudinal is composed of two or three wires varying from No. 12 gauge up to and including No. 4 wires stranded or twisted together with a long lay. These longitudinals either solid or stranded are invariably spaced 4-inch centers, the sizes being varied in order to obtain the desired cross sectional area of steel per foot of width.

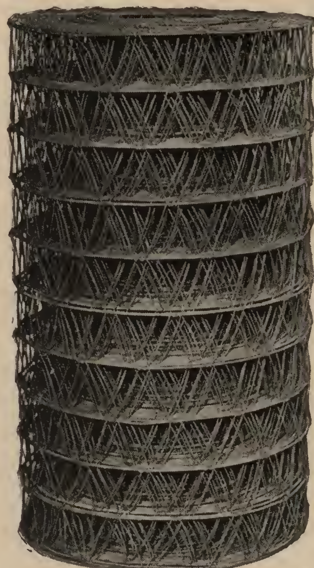
The transverse or diagonal cross wires are so woven between the longitudinals that perfect triangles are formed by their arrangement, thereby not only lending additional carrying strength to the longitudinal or tension members, but positively spacing them and providing a most perfect distribution of the steel. These diagonal cross or transverse wires are woven either 2 or 4 inches apart, as is desired. It is the most perfect reinforcement for concentrated loads, distributing the stress imposed by the load throughout the floor slab. A hinge joint is provided on each longitudinal, which enables this reinforcement to be folded longitudinally in any desired shape, making it adaptable to all kinds of concrete construction. Its design provides a most perfect mechanical bond between the steel and the concrete, and from the fact that it is not galvanized (unless specifically ordered) the maximum adhesive bond is developed.

A sufficient area of steel is provided in the cross wires of **Triangle Mesh Reinforcement** to prevent temperature cracks, thereby eliminating the necessity of laying additional reinforcement at right angles to the longitudinal or tension members.

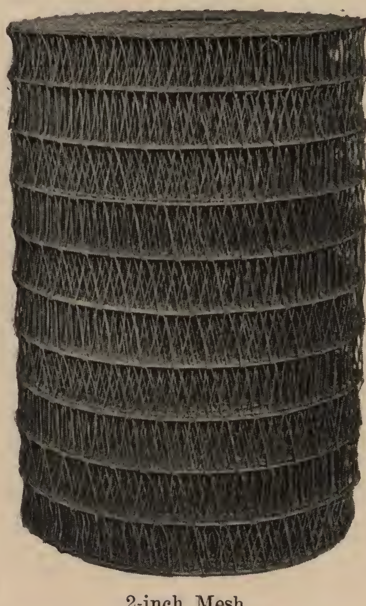


4-inch Triangle Mesh.
Concrete Reinforcement.

Patents Applied for.



4-inch Mesh.



2-inch Mesh.

Triangle Mesh Wire Reinforcement. Made in 150, 300 and 600-ft. lengths. and in 18″, 22″, 26″, 30″, 34″, 38″, 42″, 46″, 50″, 54″ and 58″ widths.

LONGITUDINALS SPACED 4-INCH CENTERS.

CROSS WIRES SPACED 4-INCH CENTERS.

Number and Gauge of Wires, Areas Per Foot Width and Weights Per 100 Square Feet.

Styles Marked * Usually Carried in Stock.

Style Number	No. of Wires Each Long	Gauge of Wire Each Long	Gauge of Cross Wires	Sectional Area Long. Sq. In.	Sectional Area Cross Wires	Cross Sectional Area per Ft. Width	Approximate Weight per 100 Sq. Ft.
* 4......	1	6	14	.087	.025	.102	43
5......	1	8	14	.062	.025	.077	34
6......	1	10	14	.043	.025	.058	27
* 7......	1	12	14	.026	.025	.041	21
*23......	1	$\frac{1}{4}''$	$12\frac{1}{2}$	.147	.038	.170	72
24......	1	4	$12\frac{1}{2}$	.119	.038	.142	62
25......	1	5	$12\frac{1}{2}$	.101	.038	.124	55
*26......	1	6	$12\frac{1}{2}$	.087	.038	.110	50
*27......	1	8	$12\frac{1}{2}$	.062	.038	.085	41
28......	1	10	$12\frac{1}{2}$	.043	.038	.066	34
29......	1	12	$12\frac{1}{2}$	.026	.038	.049	28
31......	2	4	$12\frac{1}{2}$	.238	.038	.261	106
32......	2	5	$12\frac{1}{2}$	.202	.038	.225	92
33......	2	6	$12\frac{1}{2}$	.174	.038	.196	82
34......	2	8	$12\frac{1}{2}$	.124	.038	.146	63
35......	2	10	$12\frac{1}{2}$	.086	.038	.109	50
36......	2	12	$12\frac{1}{2}$	.052	.038	.075	37
*38......	3	4	$12\frac{1}{2}$	.358	.038	.380	151
39......	3	5	$12\frac{1}{2}$	.303	.038	.325	130
40......	3	6	$12\frac{1}{2}$	.260	.038	.283	114
41......	3	8	$12\frac{1}{2}$	.185	.038	.208	87
*42......	3	10	$12\frac{1}{2}$	.129	.038	.151	66
43......	3	12	$12\frac{1}{2}$	.078	.038	.101	47

Special Sizes on Application.

LENGTH OF ROLLS: 150-ft., 300-ft. and 600-ft.

WIDTHS: 18-in., 22-in., 26-in., 30-in., 34-in., 38-in., 42-in., 46-in., 50-in., 54-in. and 58-in.

LONGITUDINAL SPACED 4-INCH CENTERS.

CROSS WIRES SPACED 2-INCH CENTERS.

Number and Gauge of Wires, Areas Per Foot Width and Weights Per 100 Square Feet.

Styles Marked * Usually Carried in Stock.

Style Number	No. of Wires Each Long.	Gauge of Wire Each Long.	Gange of Cross Wires.	Sectional Area Long. Sq. In.	Sectional Area Cross Wires. Sq. In.	Cross Sectional Area per Ft. Width.	Approximate Weight per 100 Sq. Ft.
4-A....	1	6	14	.087	.050	.102	53
5-A....	1	8	14	.062	.050	.077	44
6-A....	1	10	14	.043	.050	.058	37
* 7-A....	1	12	14	.026	.050	.041	31
23-A....	1	¼″	12½	.147	.076	.170	86
24-A....	1	4	12½	.119	.076	.142	76
25-A....	1	5	12½	.101	.076	.124	70
26-A....	1	6	12½	.087	.076	.110	64
27-A....	1	8	12½	.062	.076	.085	55
*28-A....	1	10	12½	.043	.076	.066	48
29-A....	1	12	12½	.026	.076	.049	42
31-A....	2	4	12½	.238	.076	.261	120
32-A....	2	5	12½	.202	.076	.225	107
33-A....	2	6	12½	.174	.076	.196	97
34-A....	2	8	12½	.124	.076	.146	78
35-A....	2	10	12½	.086	.076	.109	64
36-A....	2	12	12½	.052	.076	.075	52
38-A....	3	4	12½	.358	.076	.380	165
39-A....	3	5	12½	.303	.076	.325	145
40-A....	3	6	12½	.260	.076	.283	129
41-A....	3	8	12½	.185	.076	.208	101
42-A....	3	10	12½	.129	.076	.151	81
43-A....	3	12	12½	.078	.076	.101	62

Special Sizes on Application.

LENGTH OF ROLLS: 150-ft., 300-ft. and 600-ft.

WIDTHS: 18-in., 22-in., 26-in., 30-in., 34-in., 38-in., 42-in., 46-in., 50-in., 54-in. and 58-in.

Table Giving Areas in Square Feet Per Roll of

TRIANGLE MESH REINFORCEMENT.

Width of Roll in Inches.	Square Feet of Reinforcement in Roll. 150 ft. roll.	300 ft. roll.	600 ft. roll.
18	225	450	900
22	275	550	1100
26	325	650	1300
30	375	750	1500
34	425	850	1700
38	475	950	1900
42	525	1050	2100
46	575	1150	2300
50	625	1250	2500
54	675	1350	2700
58	725	1450	2900

As indicated in the above table, Triangle Mesh Reinforcement is made up in the following widths: 18, 22, 26, 30, 34, 38, 42, 46, 50, 54 and 58 inches, and in standards lengths of rolls of 150, 300 and 600 feet.

For the lighter styles, rolls of any of the above lengths may be used. Material of medium weights are recommended to be used in 150 or 300 foot lengths, while with the heaviest styles it is more conveniently handled in rolls containing 150 foot lengths.

AMERICAN STEEL & WIRE CO.'S STEEL AND IRON WIRE GAUGE AND DIFFERENT SIZES OF WIRE.

Diameter Inches.	A. S. & W. Gauge	Diameter, Inches.	Area, Sq. Inches.	Pounds per Foot.	Pounds per Mile	Feet per Pound.	Feet per 2,000 Lbs.
$\frac{1}{2}$		.500	.19635	.6625	3498.00	1.50	3018
	7/0	.490	.18857	.6363	3359.66	1.51	3023
$\frac{15}{32}$		.468	.17202	.5804	3064.51	1.72	3445
	6/0	.460	.16619	.5608	2961.02	1.78	3566
$\frac{7}{16}$		.437	.14998	.5061	2672.21	1.97	3952
	5/0	.430	.14532	.4901	2587.72	2.04	4081
$\frac{13}{32}$		.406	.12946	.4368	2306.30	2.28	4578
	4/0	.393	.12130	.4094	2161.63	2.44	4885
$\frac{3}{8}$		.375	.11044	.3726	1967.33	2.68	5367
	3/0	.362	.10292	.3473	1833.74	2.87	5758
$\frac{11}{32}$		.343	.09240	.3117	1645.78	3.20	6412
	2/0	.331	.08604	.2904	1533.31	3.44	6887
$\frac{5}{16}$		.312	.07645	.2579	1361.71	3.87	7755
	0	.307	.07402	.2497	1318.41	4.00	8011
	1	.283	.06290	.2123	1120.94	4.71	9420
$\frac{9}{32}$		.281	.06210	.2092	1104.57	4.78	9560
	2	.263	.05432	.1834	968.35	5.45	10905
$\frac{1}{4}$		.250	.04908	.1656	874.36	6.03	12077
	3	.244	.04675	.1578	733.18	6.33	12674
	4	.225	.03976	.1342	708.57	7.45	14903
$\frac{7}{32}$		.218	.03732	.1259	664.75	7.94	15885
	5	.207	.03365	.1135	559.28	8.81	17621
	6	.192	.02895	.0977	515.85	10.23	20471
$\frac{3}{16}$		.187	.02746	.0926	488.92	10.79	21598
	7	.177	.02460	.0830	438.24	12.04	24096
	8	.162	.02061	.0696	367.48	14.36	28735
$\frac{5}{32}$		.156	.01911	.0644	340.03	15.52	31056
	9	.148	.01720	.0580	306.24	17.24	34482
	10	.135	.01431	.0483	255.02	20.70	41408
$\frac{1}{8}$		.125	.01227	.0414	218.59	24.15	48309
	11	.120	.01130	.0382	201.69	26.17	52356
	12	.105	.00865	.0292	154.17	34.24	68493
$\frac{3}{32}$		.093	.00679	.0229	120.91	43.66	87336
	13	.092	.00664	.0224	118.27	44.64	89286
	14	.080	.00502	.0169	89.23	59.17	118343
	15	.072	.00407	.0137	72.33	72.99	145985
	16	.063	.00311	.0105	55.44	95.23	190476

COMPARATIVE SIZES WIRE GAUGE IN DECIMALS OF AN INCH

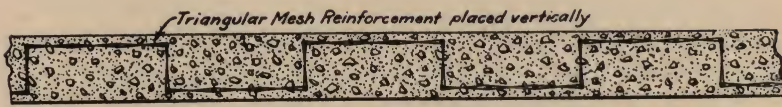
No. of Wire Gauge.	American Steel & Wire Co.	American Standard (B. & S.)	Birmingham or Stubs'.	British Imperial Standard.*	Old English or London.	French.
0000000	.4900	.....	....	.500	.....	....
000000	.4615	.58000	....	.464	.....	....
00000	.4305	.51650	.500	.432	.....	....
0000	.3938	.46000	.454	.400	.4540	....
000	.3625	.40964	.425	.372	.4250	....
00	.3310	.36480	.380	.348	.3800	....
0	.3065	.32486	.340	.324	.3400	....
1	.2830	.28930	.300	.300	.3000	.0325
2	.2625	.25763	.284	.276	.2840	.040
3	.2437	.22942	.259	.252	.2590	.050
4	.2253	.20431	.238	.232	.2380	.0625
5	.2070	.18194	.220	.212	.2200	.068
6	.1920	.16202	.203	.192	.2030	.083
7	.1770	.14428	.180	.176	.1800	.097
8	.1620	.12849	.165	.160	.1650	.110
9	.1483	.11443	.148	.144	.1480	.120
10	.1350	.10189	.134	.128	.1340	.135
11	.1205	.09074	.120	.116	.1200	.149
12	.1055	.08081	.109	.104	.1090	.162
13	.0915	.07196	.095	.092	.0950	.172
14	.0800	.06408	.083	.080	.0830	.185
15	.0720	.05706	.072	.072	.0720	.197
16	.0625	.05082	.065	.064	.0650	.212
17	.0540	.04525	.058	.056	.0580	.225
18	.0475	.04030	.049	.048	.0490	.238
19	.0410	.03589	.042	.040	.0400	.250
20	.0348	.03196	.035	.036	.0350	.263
21	.0317	.02846	.032	.032	.0315	.279
22	.0286	.02535	.028	.028	.0295	.290
23	.0258	.02257	.025	.024	.0270	.303
24	.0230	.02010	.022	.022	.0250	.316
25	.0204	.01790	.020	.020	.0230	.331
26	.0181	.01594	.018	.018	.0205	.342
27	.0173	.01420	.016	.0164	.01875	.356
28	.0162	.01264	.014	.0148	.01650	.371
29	.0150	.01126	.013	.0136	.01550	.383
30	.0140	.01003	.012	.0124	.01375	.394
31	.0132	.00893	.010	.0116	.01225	.408
32	.0128	.00795	.009	.0108	.01125	.419
33	.0118	.00708	.008	.0100	.01025	.431
34	.0104	.00630	.007	.0092	.00950	.448
35	.0095	.00561	.005	.0084	.00900	.458
36	.0090	.00500	.004	.0076	.00750	.472
37	.0085	.00445	....	.0068	.00650	.485
38	.0080	.00396	....	.0060	.00575	.499
39	.0075	.00353	....	.0052	.00500	.509
40	.0070	.00314	....	.0048	.00450	.524

*Also called New British or English Legal Standard.

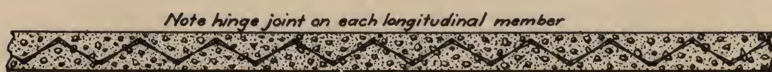
WEIGHTS AND AREAS OF SQUARE AND ROUND BARS AND CIRCUMFERENCES OF ROUND BARS.

One Cubic Foot of Steel Weighing 489.6 Lbs.

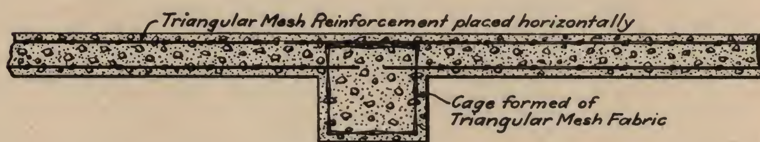
Thickness or Diameter in Inches.	Weight of Square Ber, 1 ft. Long.	Weight of Round Bar, 1 ft. Long.	Area of Square Bar in Square Inches.	Area of Round Bar in Square Inches.	Circumference of Round Bar in Inches.
0					
$\frac{1}{16}$	.013	.010	.0039	.0031	.1963
$\frac{1}{8}$	.053	.042	.0156	.0123	.3927
$\frac{3}{16}$	.119	.094	.0352	.0276	.5890
$\frac{1}{4}$	.212	.167	.0625	.0491	.7854
$\frac{5}{16}$	.333	.261	.0977	.0767	.9817
$\frac{3}{8}$	.478	.375	.1406	.1104	1.1781
$\frac{7}{16}$	.651	.511	.1914	.1503	1.3744
$\frac{1}{2}$	.850	.667	.2500	.1963	1.5708
$\frac{9}{16}$	1.076	.845	.3164	.2485	1.7671
$\frac{5}{8}$	1.328	1.043	.3906	.3068	1.9635
$\frac{11}{16}$	1.608	1.262	.4727	.3712	2.1598
$\frac{3}{4}$	1.913	1.502	.5625	.4418	2.3562
$\frac{13}{16}$	2.245	1.763	.6602	.5185	2.5525
$\frac{7}{8}$	2.603	2.044	.7656	.6013	2.7489
$\frac{15}{16}$	2.989	2.347	.8789	.6903	2.9452
1	3.400	2.670	1.0000	.7854	3.1416
$\frac{1}{16}$	3.838	3.014	1.1289	.8866	3.3379
$\frac{1}{8}$	4.303	3.379	1.2656	.9940	3.5343
$\frac{3}{16}$	4.795	3.766	1.4102	1.1075	3.7306
$\frac{1}{4}$	5.312	4.173	1.5625	1.2272	3.9270
$\frac{5}{16}$	5.857	4.600	1.7227	1.3530	4.1233
$\frac{3}{8}$	6.428	5.049	1.8906	1.4849	4.3197
$\frac{7}{16}$	7.026	5.518	2.0664	1.6230	4.5160
$\frac{1}{2}$	7.650	6.008	2.2500	1.7671	4.7124
$\frac{9}{16}$	8.301	6.520	2.4414	1.9175	4.9087
$\frac{5}{8}$	8.978	7.051	2.6406	2.0739	5.1051
$\frac{11}{16}$	9.682	7.604	2.8477	2.2365	5.3014
$\frac{3}{4}$	10.41	8.178	3.0625	2.4053	5.4978
$\frac{3}{16}$	11.17	8.773	3.2852	2.5802	5.6941
$\frac{7}{8}$	11.95	9.388	3.5156	2.7612	5.8905
$\frac{15}{16}$	12.76	10.02	3.7539	2.9483	6.0868



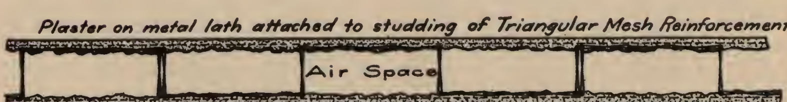
Wall



Wall

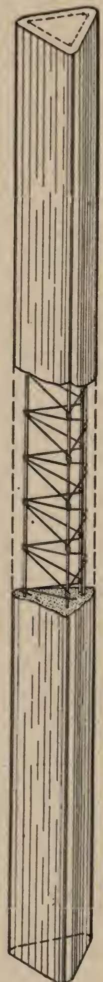


Wall with Pilaster

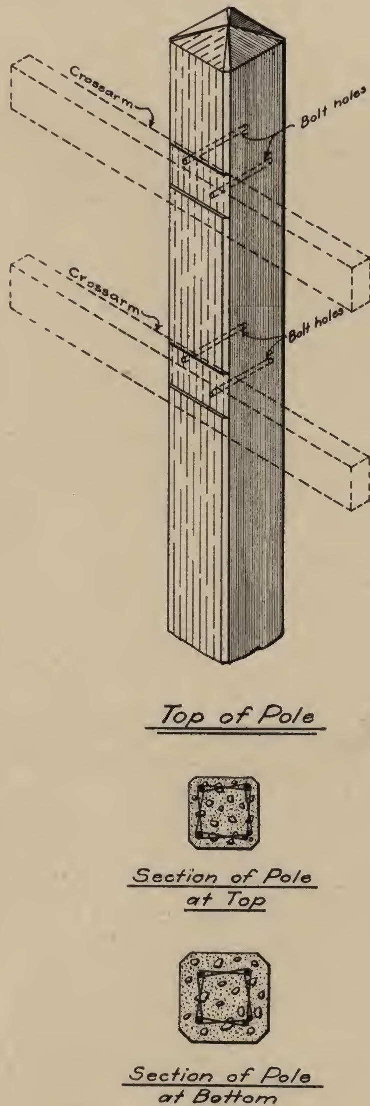


Plaster Partition Wall

Wall Construction.

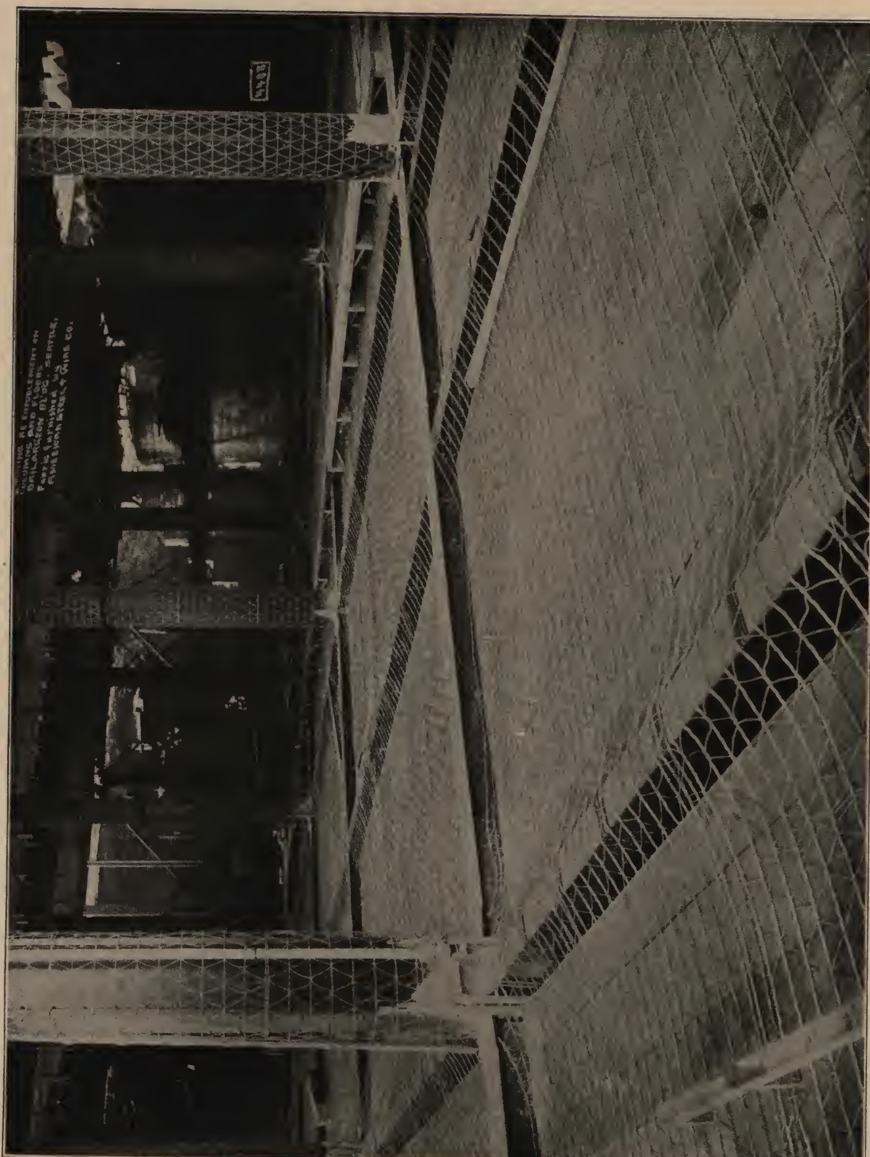


Fence Post



Telegraph Pole

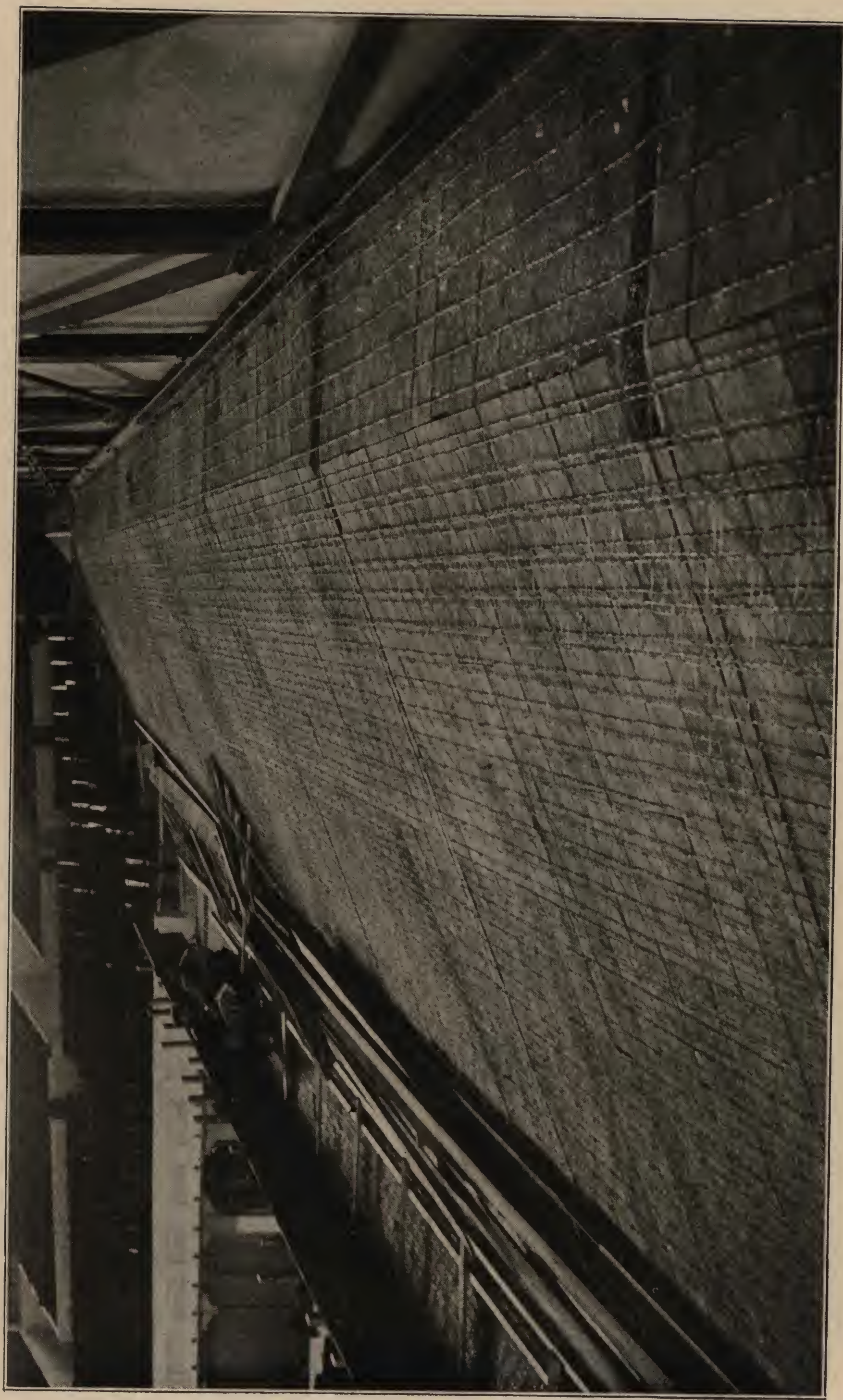
Reinforced with 2-inch Triangle Mesh Reinforcement.



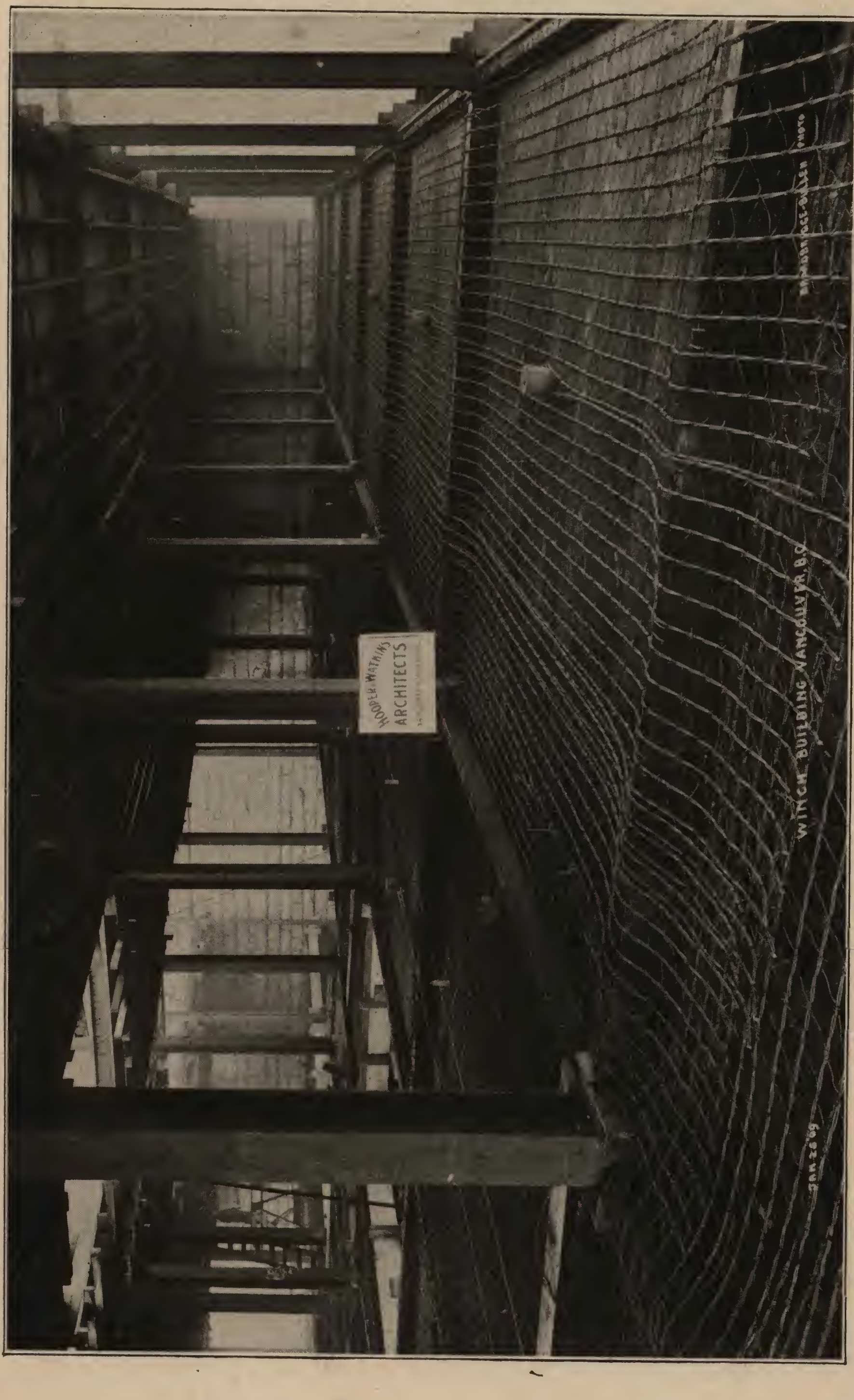
SHOWING CONSTRUCTION USED IN BALLARGEON BUILDING, SEATTLE, WASH.
American Steel & Wire Co.'s Triangle Mesh Reinforcement.

HENRY BUILDING, SEATTLE, WASH.

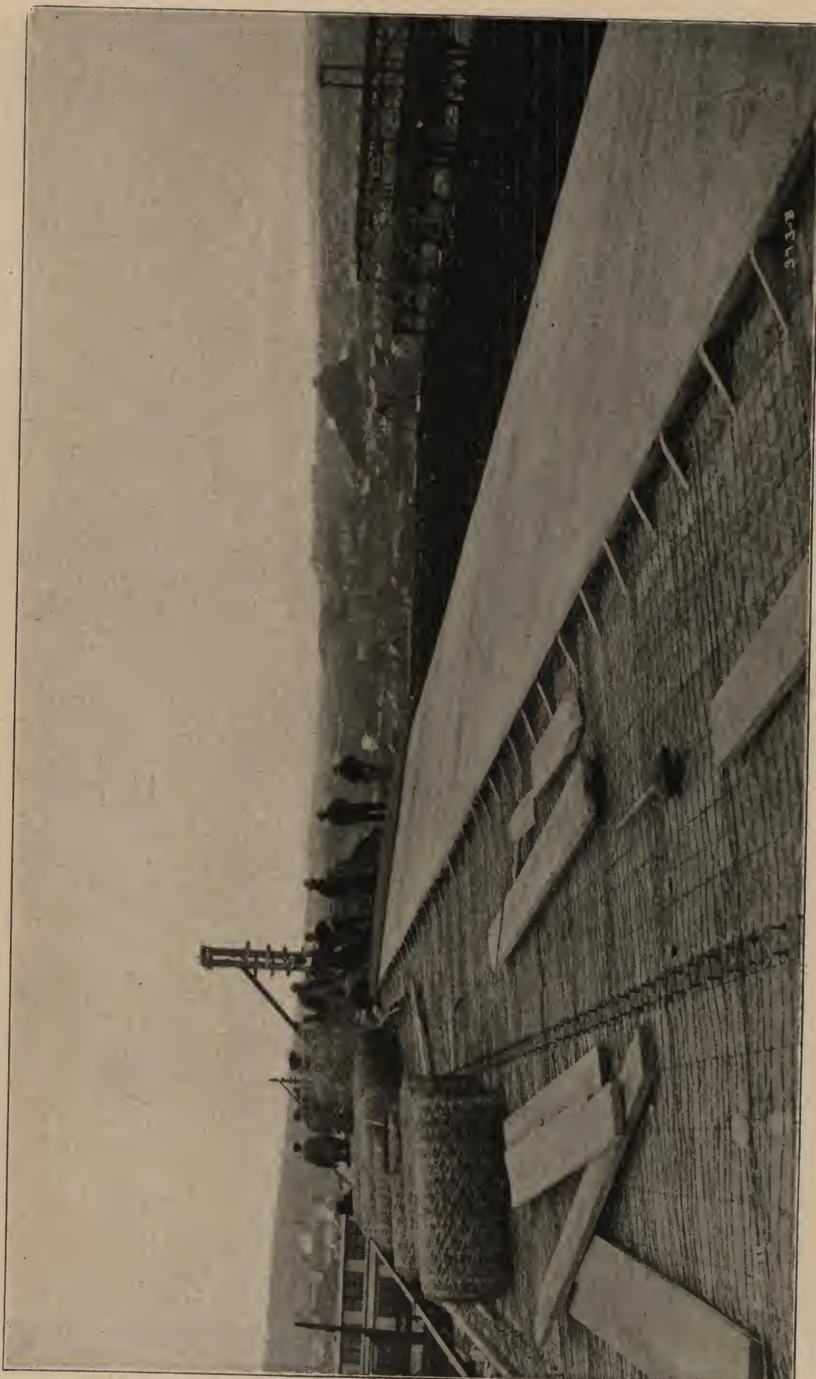
Erected for Metropolitan Bldg. Co. by Stone & Webster Engr. Corporation. Howell & Stocks, Supervising Architects.



DENVER AUDITORIUM, DENVER, COLO.
Showing American Steel & Wire Co.'s Triangle Mesh Reinforcement as Used in Balcony Floors.



WINCH BUILDING, VANCOUVER, BRITISH COLUMBIA.
Hooper & Watkins, Architects.



Showing Triangle Mesh Reinforcement as Used in the Roof of the D., L. & W. Repair Shop, Scranton, Pa.

Showing Triangle Mesh Reinforcement as Used in Reinforced Concrete Highway-Bridge Floors.

Reinforced Concrete Fences, Used by Union Stock Yard & Transit Co., Chicago.
Reinforced with American Steel & Wire Co.'s Triangle Mesh.

REINFORCED CONCRETE UNLOADING PLATFORM AT UNION STOCK YARD & TRANSIT CO., CHICAGO.
Piece Construction Reinforced with American Steel & Wire Co.'s Triangle Mesh.

Showing Reinforced Concrete Piece Constructed Columns and Girders,
Being Erected in Place by Union Stock Yard & Transit Co., Chicago. American Steel & Wire Co.'s Triangle Mesh Used.

TEST No. 60 PRINTED REPORT No. 9

REPORT

OF A

FIRE, LOAD and WATER TEST

MADE UPON A

TRIANGULAR REINFORCED CONCRETE FLOOR SYSTEM

CONSTRUCTED BY

AMERICAN STEEL AND WIRE COMPANY

AT THE

FIRE TESTING STATION
COLUMBIA UNIVERSITY, NEW YORK CITY

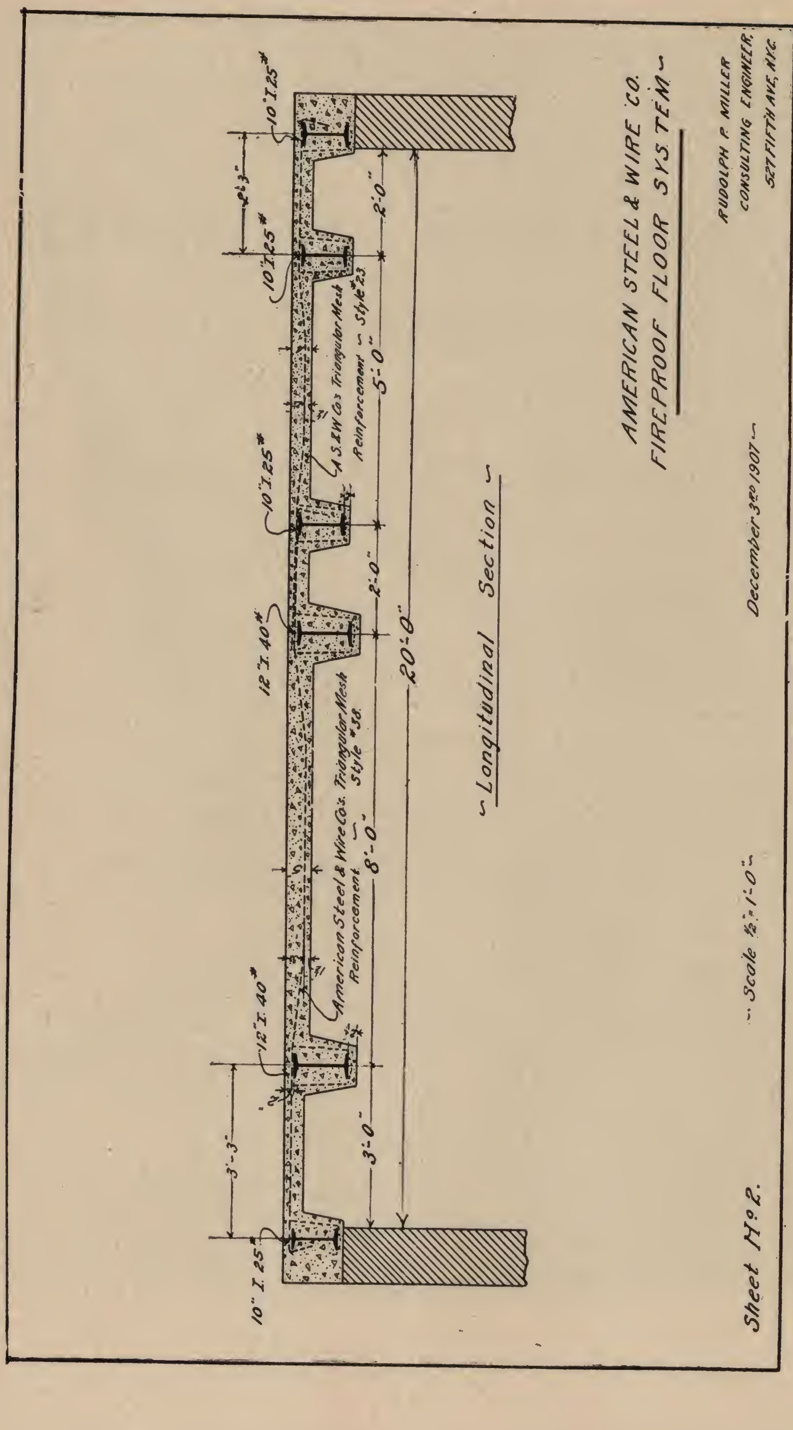
TEST CONDUCTED BY

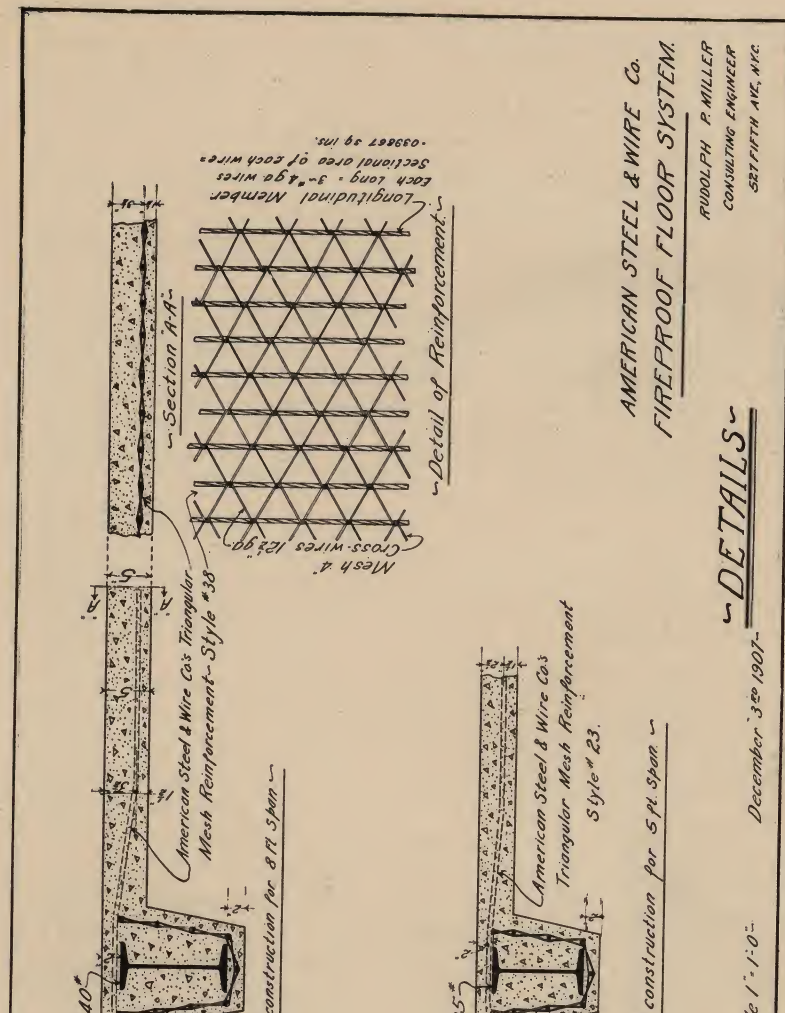
IRA H. WOOLSON, E. M.
Adjunct Professor of Civil Engineering

IN CO-OPERATION WITH

THE CITY BUILDING BUREAUS

NEW YORK, MARCH 5, 1908


AMERICAN STEEL & WIRE CO.
FIREPROOF FLOOR SYSTEM
RUDOLPH P. MILLER
CONSULTING ENGINEER.
527 FIFTH AVE, N.Y.C.
December 3rd 1907
Longitudinal Section
Scale ½" = 1'-0"
Sheet No. 2.
A.S.&W. Co's Triangular Mesh Reinforcement Style #23
American Steel & Wire Co's. Triangular Mesh Reinforcement Style #38.
10" I. 25#
12" I. 40#
20'-0"
8'-0"
5'-0"
3'-0"
2'-0"
3'-3"



AMERICAN STEEL & WIRE Co.
FIREPROOF FLOOR SYSTEM.
RUDOLPH P. MILLER
CONSULTING ENGINEER
527 FIFTH AVE, N.Y.C.
~DETAILS~
December 3rd 1907
Scale 1" = 1'-0"
Sheet No 3
Section 'A-A'
Detail of Reinforcement
Longitudinal Member
Each long = 3-#4 ga. wires
Sectional area of each wire = .039867 sq. ins.
Mesh 4"
Cross-wires 12½ ga.
12" I-40#
A.S.&W. Co's Triangular Mesh Reinforcement - Style #8
American Steel & Wire Co's Triangular Mesh Reinforcement - Style #38
Detail of construction for 8 ft. Span
10" I-25#
A.S.&W. Co's Triangular Mesh Reinforcement - Style #8
American Steel & Wire Co's Triangular Mesh Reinforcement Style #23.
Detail of construction for 5 ft. Span

METHOD OF CONSTRUCTION.

The test was conducted in test house No. 2, which is a reinforced cinder concrete structure 14′ x 20′ on the inside. It is supplied with six suitable chimneys at the top and draft openings thru the wall at the bottom. The fire grate is located 2′ — 6″ above the ground level.

The floor construction under test formed the temporary roof of the test house. The under side or ceiling being 9′ — 6″ above the grate.

Two types of floor arches were tested, one with a span of eight feet and the other a span of five feet.

The eight foot span occupied the south side of the test chamber and was made of cinder concrete five inches thick, reinforced with the metal fabric of the American Steel & Wire Company, known as Style No. 38. The disposition of which is indicated in the attached blueprints and photograph No. 1. This metal fabric consisted of three twisted strands of No. 4 steel wire, spaced and secured every four inches by No. 12½ diagonal steel cross wire.

The five-foot span occupied the north side of the test chamber and was made of cinder concrete four inches thick, reinforced with American Steel & Wire Company's fabric, Style No. 23. The metal fabric in this span consisted of one strand of one-quarter-inch steel wire, spaced and secured every four inches by diagonal wires the same as in the fabric of the eight-foot span. The character of this fabric can be seen in the middle of photo. No. 1, where it laps over the two central beams. The fabric in the spans was placed 1½ inches above the bottom surface of the concrete.

The eight-foot span was supported by 12″ — 40# I beams, and the five-foot span by 10″ — 25# I beams. These beams were protected by two inches of concrete below the bottom flange of the beams, and one and one-half inches of concrete at each side of the lower flanges.

The concrete was held in place by American Steel & Wire Company's fabric, Style No. 8, consisting of one strand of No. 12½ wire secured in every four inches by No. 14 diagonal cross wires.

The concrete used in the construction was a 1-2-5 mixture of Portland cement, clean sharp sand, and boiler cinders.

Specimen blocks of this mixture were made at the time that the material was being placed, and were tested for strength when about eleven weeks old. They gave an average value of 1,000 lbs. per sq. inch.

Full details of the construction are given in the attached drawings and photographs.

The five-foot span was put in place December 18, 1907, and the eight-foot span on January 25, 1908. The former was therefore about eleven weeks old, and the latter six weeks at test.

Owing to the fact that it was stormy and freezing weather most of the period the concrete was setting, a salamander was kept burning in the test building the greater portion of the time.

Purpose of the Test.

The purpose of the test was to determine the effect of a continuous fire below the floor for four hours at an average temperature of 1700° F., the floor carrying at the same time a distributed load of 150 lbs. per sq. ft.; at the end of the four hours the under side of the floor (or ceiling) while still red hot to be subjected to a 1⅛″ stream of cold water at short range, thru 100′ of hose under 60 lbs. pressure at the nozzle for five minutes, then the upper side of the floor (which forms the roof of the test building) to be flooded at low pressure; afterwards the stream to be again applied at full pressure on the under side for five minutes longer.

Deflection of beams and floor to be measured continuously during the test. The load then to be removed and after the floor becomes cold, it shall be reloaded to 600 lbs. per sq. ft. and deflections noted.

Temperatures.

The temperatures of the fire were obtained by three electric pyrometer couples suspended thru the floor from above and hanging about 6″ below the ceiling. The location of the couples is indicated on the temperature curve sheet.

Readings were made upon each couple every three minutes. The fuel used was dry cord wood, the frequency of firing being determined by the temperature of the test chamber. The "Log of Temperature Readings," together with plotted curve for the middle couple, is herewith attached.

Deflections.

The deflections which occurred during the test were measured by a Y level, reading upon rods located upon the ends and middle of each beam, also upon the middle of the floor arches. The "Table of Deflections" gives full information regarding the variation in level of each point thruout the test, and the "Deflection Diagram" shows the relative position of the three members graphically at different times.

Water.

The water was applied by firemen with an engine detailed from 143rd Street and 8th Avenue fire station, under direction of Battalion Chief Edward S. Root. The pressure gauge was carefully watched, and 60 lbs. maintained at the nozzle.

In applying the water, the stream was thrown back and forth over the whole ceiling as much as possible, and not allowed to strike continuously on one spot. After five minutes' application the pressure was

reduced to about 30 lbs. and the top of the floor flooded, then the stream was applied to the ceiling again at full pressure for five minutes longer. Total time of application at full pressure was 10 minutes.

Thru an error the firemen brought with them a $1\frac{1}{4}$-inch nozzle instead of the regulation $1\frac{1}{8}$-inch diameter. This larger nozzle was therefore used, which made the stream somewhat more severe than usual.

Results of the Test.

About one hour after starting the fire three cracks appeared on the top of the floor or roof of the building. Two of these cracks were parallel with and nearly over the two supporting beams of the eight-foot span, and the third was similarly located along the middle beam of the five-foot span. These cracks gradually opened during the test to a maximum width of about $\frac{3}{32}$ of an inch. Some water boiled out of the concrete and ran out upon the roof. During the latter part of the test, a few small diagonal cracks developed near the ends of the beams, and ran across the floor slabs towards the middle of the spans.

All of these cracks were evidently due to expansion. They appeared to be on the upper surface only. The underside of the floor or ceiling of the test chamber was in perfect condition so far as could be observed up to the application of water. The water scored the surface somewhat in front of the doors where its action was most severe, but only to a depth of about one inch as a maximum. Considerable of the concrete thus washed out was along the line of the cracks between the centering boards where the cement had evidently been carried away by the escaping water at the time the floor was placed. This is a defect that could be easily avoided.

The wire mesh encasing the bottom of the middle I beam of the eight-foot span was exposed for about three feet near the middle of the beam, and the flange of the beam itself was bare for eighteen inches. This was the only metal exposed by the application of water.

There was one crack four feet long in the central portion of the eight-foot span about one foot from the middle beam and running parallel with it.

There were several small cracks across the bottom of the beams, but the concrete remained firm and secure.

As a whole the ceiling was in excellent condition at the completion of the test.

The maximum deflection at the middle of the eight and five-foot span during the fire was $1\frac{1}{32}$ and $\frac{11}{16}$ inches, respectively. The larger part of this was recovered when the floor cooled, and the application of the final load of 600 lbs. per sq. foot increased the deflection on the larger span only $\frac{3}{16}$ inch, and on the smaller span $\frac{1}{16}$ inch.

No new cracks developed as a result of the loading, and there was no evidence that the floor was distressed.

The total permanent deflection of the two spans after entire removal of load was $\frac{3}{8}$ inch for the eight-foot span and $\frac{1}{4}$ inch for the five-foot span.

The test was conducted in co-operation with the Bureau of Buildings of this city, and the installation of the floor was observed by representatives from the bureaus in the various boroughs, also from Philadelphia.

The test was witnessed by the following Building Bureau Engineers:

A. Schwartz, Borough of Manhattan.
H. Vanatta, Borough of Bronx.
Harry Brown, Borough of Richmond.
W. G. Button, Bureau of Building Inspection, Philadelphia, Pa.
Spencer B. Hopkins, Bureau of Buildings, Providence, R. I.

The American Steel & Wire Company were represented by:

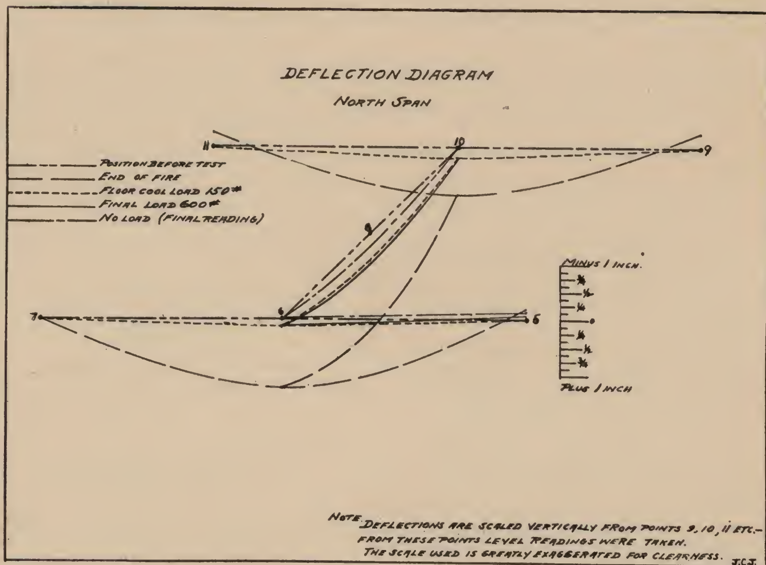
T. H. Taylor, New York Office.
H. S. Doyle, Chicago Office.
L. A. Dietrich, New York Office.
R. H. Pratt, New York Office, and several others.

A large number of spectators were present, among them being the following:

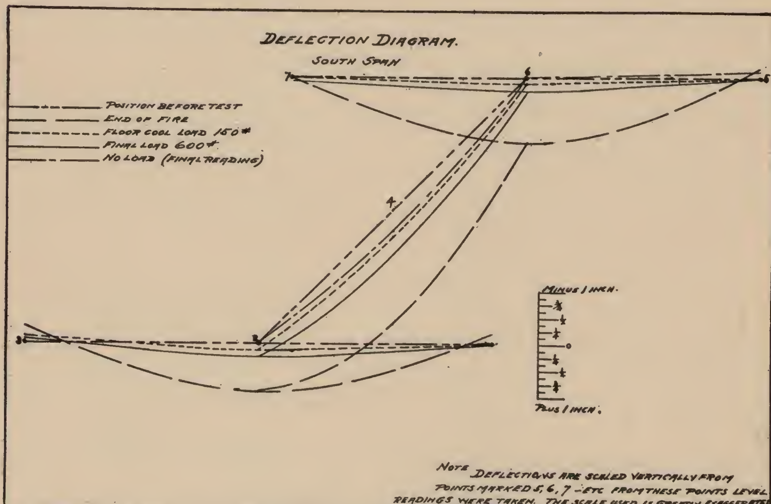
Rudolph P. Miller, Consulting Engineer, 527 5th Avenue, New York.
Colonel Hodges, Chief Engineer, Isthmian Canal Commission, Washington, D. C.
R. Allworth, Assistant Chief Engineer, Washington, D. C.
Major Langfitt, U. S. Engineering Corps, Washington, D. C.
G. H. Stewart, Rep. "Ins. Engineer," 120 Liberty Street, New York.
G. E. Stecher, Rep. "Ins. Press," 56 Cedar Street, New York.
H. R. Leonard, Engineer, Penn. R. R., Philadelphia, Pa.
J. F. Murray, Engineer, Penn. R. R., Philadelphia, Pa.
B. H. Davis, Assist. Engineer, D. L. & W. R. R., Hoboken, N. J.
J. B. French, Engineer, L. I. R. R., Jamaica, L. I., N. Y.
R. W. How, Inspector, L. I. R. R., Jamaica, L. I., N. Y.
Geo. P. Enke, Inspector, German-American Insurance Co., New York.
Frank B. Gilbreth, Cont. Eng., 34 West 26th Street, New York.
P. H. Bevier, Engineer, National Fireproofing Co., New York.
S. P. Waldron, American Bridge Co., 42 Broadway, New York.
W. H. Olthoff, U. S. Steel Prod. Exp. Co., 24 State Street, New York.
J. Traverse, Neuchatel Asphalt Co., 265 Broadway, New York.
E. Merrick, Merrick Fireproof Co., 11 Broadway, N. Y.
P. E. Bertin, 30 West 33rd Street, New York.
W. B. Ruggles, Curtin-Ruggles Co., 39 Cortlandt Street, New York.
L. R. Christie, Curtin-Ruggles Co., 39 Cortlandt Street, New York.
Capt. J. F. Shea, Jerome Avenue and 183d Street, New York.
Fuller Claflin, Architect, 1440 Broadway, New York.
J. P. H. Perry, Turner Construction Co., 11 Broadway, New York.
C. U. Carpenter, Pres. Herring-Hall-Marvin Safe Co., 400 Broadway, New York.
J. G. Ellendt, J. G. Ellendt Co., 1 Madison Avenue, New York.
W. H. Wheelock, Douglass Robinson Co., 146 Broadway, New York.
G. E. Walsh, 555 West 183d Street, New York.

Respectfully submitted,

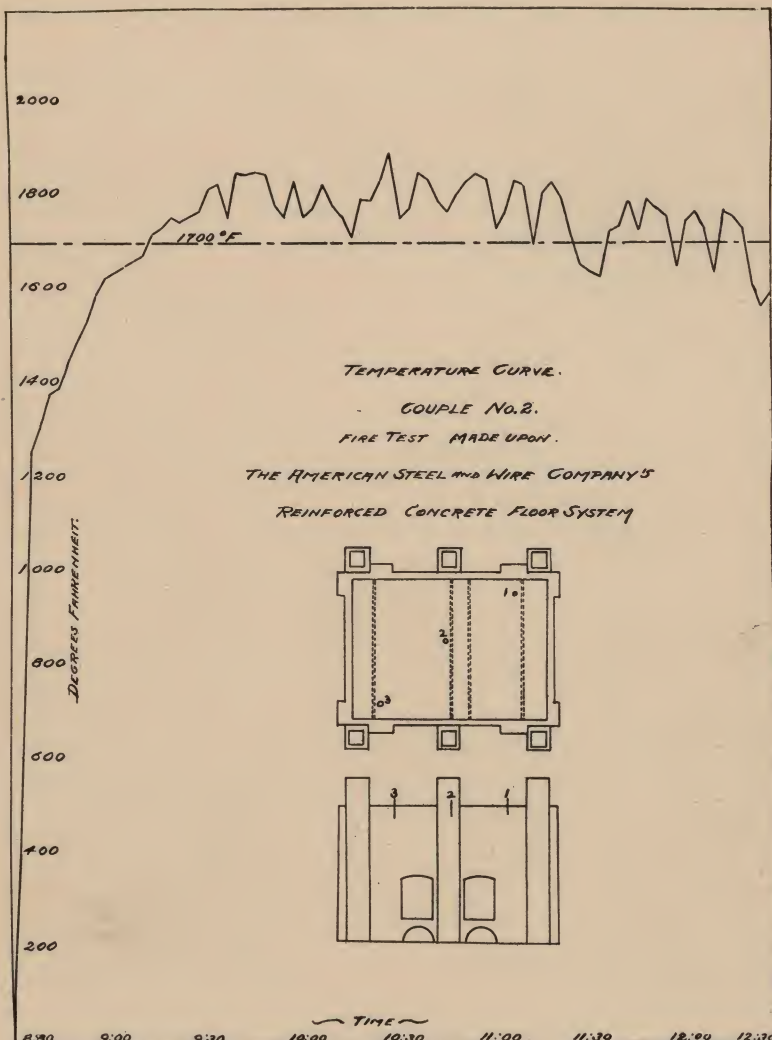
IRA H. WOOLSON.



Deflection Diagram for 5-Foot Span.



Deflection Diagram for 8-Foot Span.


2000
1800
1700°F
1600
1400
1200
1000
800
600
400
200
DEGREES FAHRENHEIT.
TEMPERATURE CURVE.
COUPLE No. 2.
FIRE TEST MADE UPON.
THE AMERICAN STEEL AND WIRE COMPANY'S
REINFORCED CONCRETE FLOOR SYSTEM
1
2
3
TIME
8:30
9:00
9:30
10:00
10:30
11:00
11:30
12:00
12:30

DEFLECTION READINGS.

NOTE—For location of rods see "Deflection Diagrams." Deflections are given in sixteenths of an inch; those preceded by a minus sign are rises, those by no sign are falls.

Deflections read by J. C. Jenkins.

	1	2	3	4	5	6	7	8	9	10	11
March 3, 1908.											
No Load.											
10.00 a. m.	0	0	0	0	0	0	0	0	0	0	0
March 4. Load, 150 lbs. per sq. ft. Start of fire.											
8.30 a. m.	0	0	—1	0	0	1	0	0	0	0	0
50 "	0	2	—3	4	—2	2	0	3	—1	0	0
9.20 "	—1	3	—3	7	—3	2	—1	4	—2	2	—1
50 "	—1	3	—4	10	—3	2	—2	7	—2	3	—1
10.20 "	—1	4	—4	15	—3	6	—2	12	—2	4	—2
50 "	—2	7	—4	19	—3	9	—2	16	—3	8	—2
11.20 "	—2	10	—4	26	—3	14	0	21	—3	11	—3
50 "	—3	13	—5	30	—3	17	0	24	—3	13	—3
12.30 "	—3	15	—5	34	—3	20	0	28	—4	14	—4
End of fire.											
March 5. Floor cool; load, 150 lbs. per sq. ft.											
8.45 a. m.	0	2	—2	8	0	2	0	7	0	3	0
Load, 600 lbs. per sq. ft. on 5′ span.											
1.30 "	0	2	—2	7	—1	2	0	8	0	3	0
Load, 600 lbs. per sq. ft. on 8′ span.											
2.40 "	0	4	—1	13	0	4	1	7	2	3	2
No load.											
4.30 "	0	0	0	6	—2	0	0	4	0	0	0

CORRECTED TOTAL DEFLECTIONS.

For Middle of Beam, Taking Into Account the Rise of Ends of Beams.

8′ Span.

	Rod 2. South Beam.	Rod 4. Centre Floor Slab.	Rod 6. Centre Beam.
March 4. End of fire, 12.30 p. m.	$1\frac{3}{16}''$	$1\frac{1}{32}''$	$1\frac{11}{32}''$
March 5. Floor cool; load, 150 lbs. per sq. ft. 8.45 a. m.	$\frac{3}{16}''$	$\frac{3}{8}''$	$\frac{1}{8}''$
Load, 600 lbs. per sq. ft. 2.40 p. m.	$\frac{9}{32}''$	$\frac{9}{16}''$	$\frac{7}{32}''$
No. load. 4.30 p. m.	$0''$	$\frac{3}{8}''$	$\frac{1}{16}''$

5′ Span.

	Rod 6. Centre Beam.	Rod 8. Centre Floor Slab.	Rod 10. North Beam.
March 4. End of fire. 12.30 p. m.	$1\frac{11}{32}''$	$\frac{11}{16}''$	$1\frac{1}{8}''$
March 5. Floor cool; load, 150 lbs. per sq. ft. 8.45 a. m.	$\frac{1}{8}''$	$\frac{9}{32}''$	$\frac{3}{16}''$
Load, 600 lbs. per sq. ft. 1.30 p. m.	$\frac{7}{32}''$	$\frac{11}{32}''$	$\frac{3}{16}''$
No load. 4.30 p. m	$\frac{1}{16}''$	$\frac{1}{4}''$	$0''$

NOTE—The results are obtained as follows:

For all beam deflections when the ends rise and the middle falls, add the middle deflections to the mean of the two end rises. When the ends and middle fall, subtract.

LOG OF TEMPERATURE READINGS.

FIRE TEST.

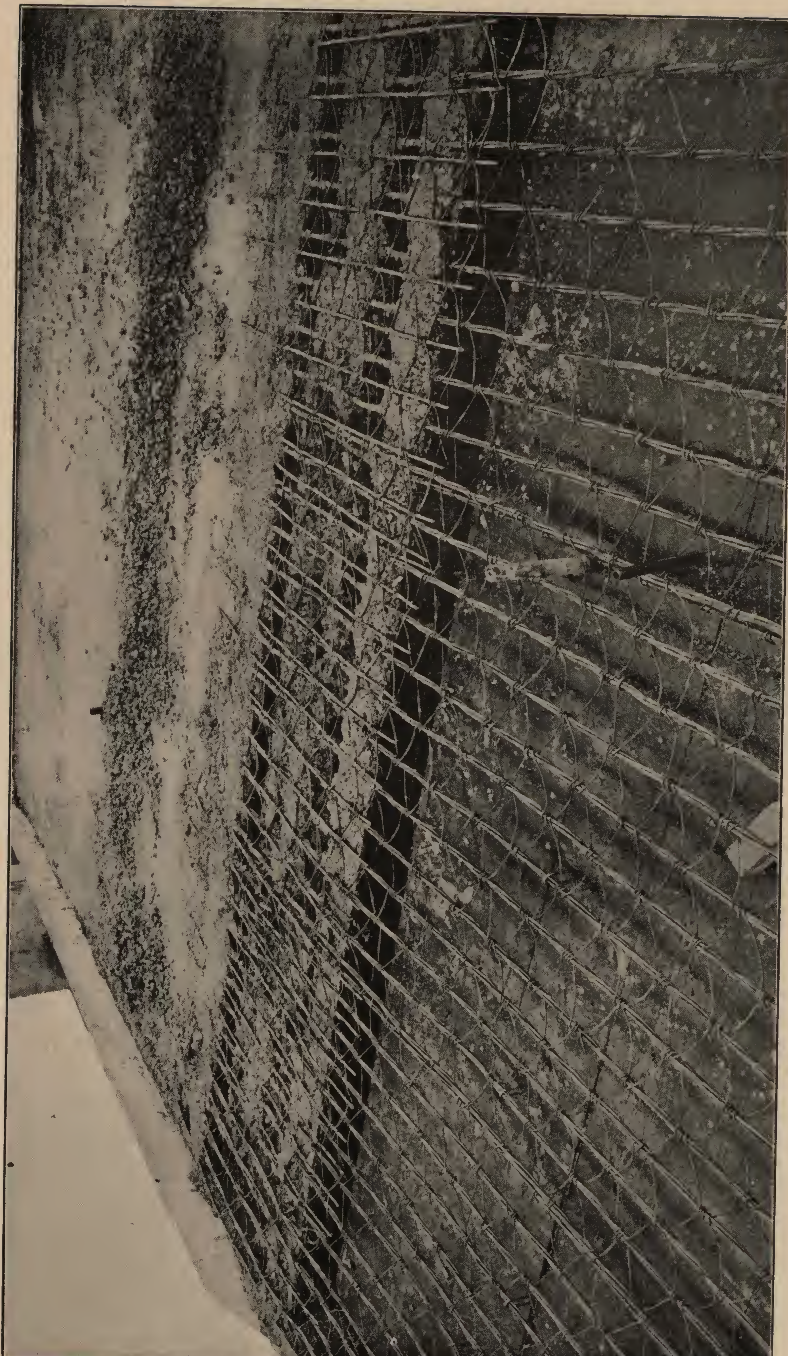
THE AMERICAN STEEL & WIRE COMPANY.

FIREPROOF FLOOR CONSTRUCTION.

Temperatures Read by J. S. Macgregor, M. S.

Time.	Couple No. 1.	Couple No. 2.	Couple No. 3.
8.30 Start.			
33	205	394	261
36	815	1259	828
39	935	1311	1130
42	1019	1380	1254
45	1156	1397	1311
48	1248	1457	1324
51	1346	1494	1374
54	1385	1530	1415
57	1447	1586	1397
9.00	1524	1622	1415
03	1562	1635	1426
06	1584	1647	1462
09	1610	1659	1499
12	1635	1671	1545
15	1659	1719	1562
18	1647	1731	1573
21	1719	1755	1597
24	1719	1743	1586
27	1707	1755	1622
30	1719	1767	1635
33	1779	1814	1659
36	1802	1825	1719
39	1732	1756	1708
42	1768	1850	1768
45	1838	1844	1774
48	1844	1850	1792
51	1780	1844	1768
9.54	1720	1780	1708
57	1732	1756	1690
10.00	1804	1827	1756
03	1744	1756	1732
06	1756	1774	1744
09	1780	1821	1804
12	1768	1780	1756
15	1732	1738	1696
18	1684	1708	1732

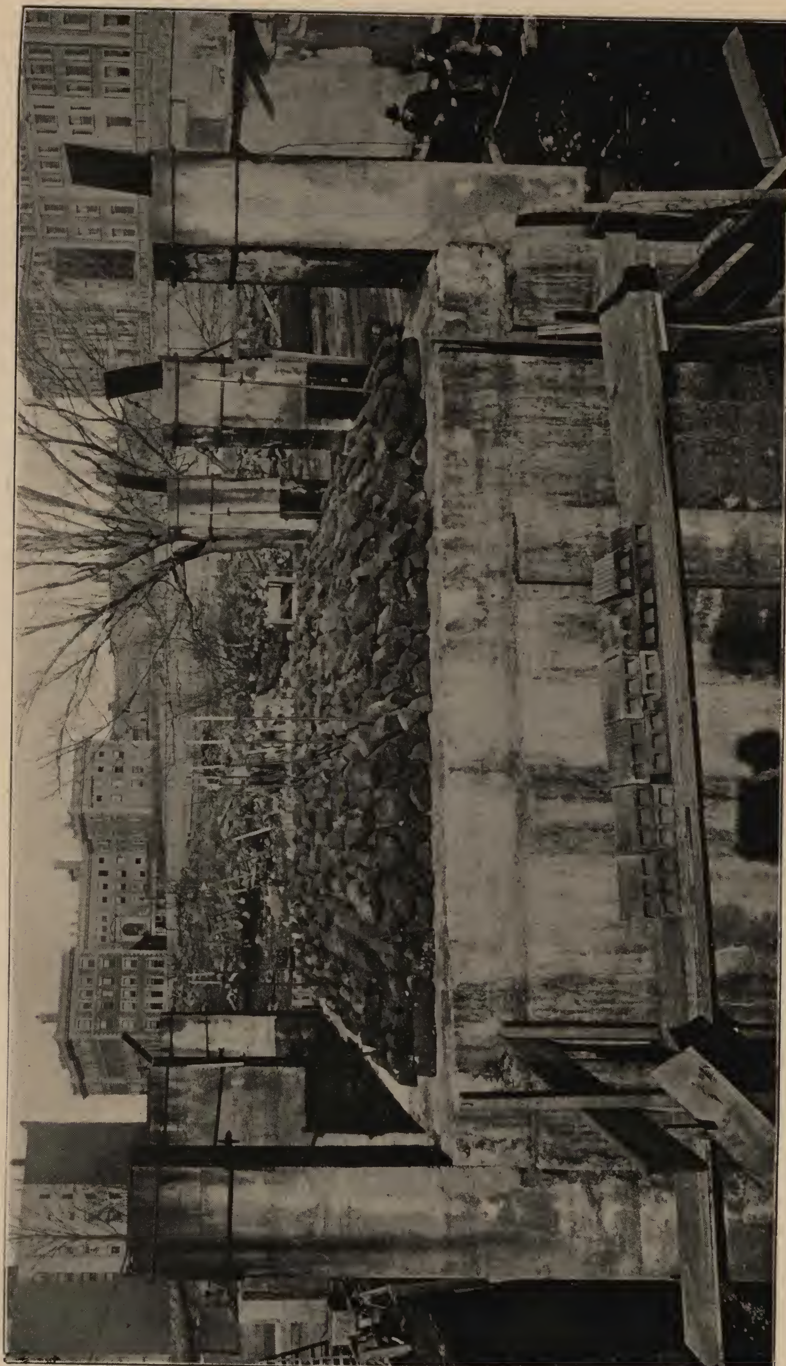
Time.	Couple No. 1.	Couple No. 2.	Couple No. 3.
21	1756	1792	1744
24	1741	1789	1717
27	1836	1830	1765
30	1857	1895	1765
33	1707	1753	1693
36	1765	1777	1717
39	1847	1853	1824
42	1830	1836	1813
45	1753	1789	1741
48	1741	1765	1729
51	1847	1801	1777
54	1819	1830	1824
57	1836	1847	1830
11.00	1824	1836	1813
03	1705	1729	1671
06	1777	1765	1729
09	1789	1836	1777
12	1777	1824	1789
15	1705	1693	1771
18	1795	1807	1747
21	1819	1830	1783
24	1771	1795	1771
27	1687	1711	1650
11.30	1638	1650	1625
33	1640	1638	1644
36	1640	1625	1644
39	1687	1723	1675
42	1711	1735	1717
45	1759	1795	1771
48	1675	1723	1675
51	1771	1795	1723
54	1747	1771	1723
57	1759	1759	1711
12.00	1612	1648	1601
03	1745	1743	1687
06	1757	1769	1745
09	1721	1732	1697
12	1600	1635	1622
15	1757	1769	1757
18	1740	1758	1734
21	1704	1728	1692
24	1618	1607	1579
27	1629	1562	1593
30	1608	1593	1580
Averages	1655	1704	1644



ARRANGEMENT OF METAL IN THE FLOOR.
Concrete Partially in Place on the 5-Foot Span. 8-Foot Span in the Foreground.

METHOD OF ANCHORING METAL AT EDGE OF FLOOR.
Concrete in Place on 8-Foot Span.

Underside of Floor Before Test, Showing Two Middle Beams, and 8-Foot Span.



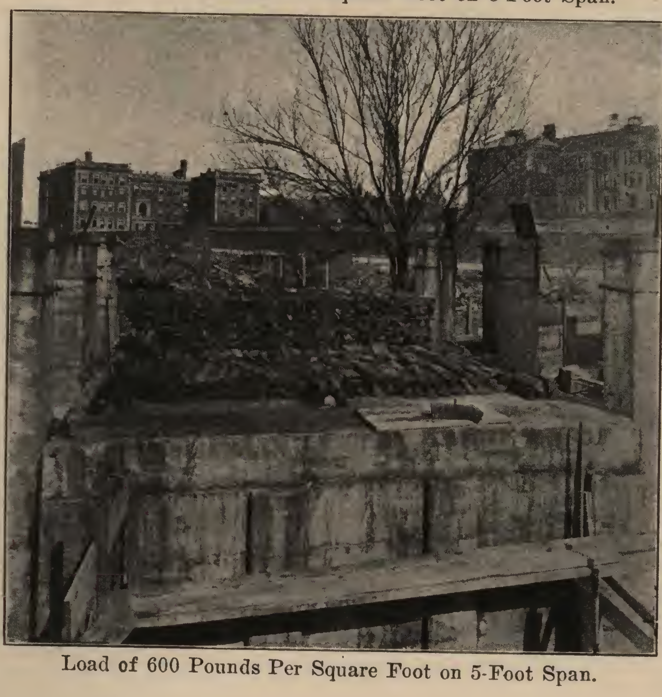
Load of 150 Pounds Per Square Foot Carried by Floor During Fire.

Building During the Fire.

Firemen Applying Water.

UNDERSIDE OF FLOOR AFTER TEST.
Condition of 8-Foot Span Shown at Top of Picture. Beams in the Middle, and 5-Foot Span Beyond.

Load of 600 Pounds Per Square Foot on 8-Foot Span.



Load of 600 Pounds Per Square Foot on 5-Foot Span.

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